A Laboratory Investigation into the Effects of Aggregate-related Factors of Critical VMA in Asphalt Paving Mixtures

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A LABORATORY INVESTIGATION INTO THE EFFECTS OF AGGREGATE-RELATED FACTORS OF CRITICAL VMA IN ASPHALT PAVING MIXTURES

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TABLE OF CONTENTS

LIST OF FIGURES

LIST OF TABLES

ABSTRACT

This report summarizes research conducted at Iowa State University on behalf of the Iowa Department of Transportation, focusing on the volumetric state of hot-mix asphalt (HMA) mixtures as they transition from stable to unstable configurations. This has traditionally been addressed during mix design by meeting a minimum voids in the mineral aggregate (VMA) requirement, based solely upon the nominal maximum aggregate size without regard to other significant aggregate-related properties. The goal was to expand the current specification to include additional aggregate properties, e.g., fineness modulus, percent crushed fine and coarse aggregate, and their interactions. The work was accomplished in three phases: a literature review, extensive laboratory testing, and statistical analysis of test results.

The literature review focused on the history and development of the current specification, laboratory methods of identifying critical mixtures, and the effects of other aggregate-related factors on critical mixtures.

The laboratory testing involved three maximum aggregate sizes (19.0, 12.5, and 9.5 millimeters), three gradations (coarse, fine, and dense), and combinations of natural and manufactured coarse and fine aggregates. Specimens were compacted using the Superpave Gyratory Compactor (SGC), conventionally tested for bulk and maximum theoretical specific gravities and physically tested using the Nottingham Asphalt Tester (NAT) under a repeated load confined configuration to identify the transition state from sound to unsound.

The statistical analysis involved using ANOVA and linear regression to examine the effects of identified aggregate factors on critical state transitions in asphalt paving mixtures and to develop predictive equations.

The results clearly demonstrate that the volumetric conditions of an HMA mixture at the stable-unstable threshold are influenced by a composite measure of the maximum aggregate size and gradation and by aggregate shape and texture. The currently defined VMA criterion, while significant, is seen to be insufficient *by itself* to correctly differentiate sound from unsound mixtures. Under current specifications, many otherwise sound mixtures are subject to rejection solely on the basis of failing to meet the VMA requirement. Based on the laboratory data and statistical analysis, a new paradigm to volumetric mix design is proposed that explicitly accounts for aggregate factors (gradation, shape, and texture).

1 INTRODUCTION

In the analysis and design of asphalt mixtures, consideration of the contributions of the three material components to the total volume of compacted mixtures has been recognized as a significant factor. The study of the component volumetric makeup of asphalt mixtures has come to be known as "volumetrics." In the simplest approach, a compacted asphalt mixture may be resolved to the individual volumes of the mineral aggregate, V_s, the asphalt binder, V_b, and the entrapped air, V_a. However, because of the inevitable characteristic of aggregate absorption by which a portion of the asphalt binder is taken *into* the body of the aggregate, the sum of the individual component volumes exceeds the total volume of a compacted asphalt mixture. As a result, two secondary volumetric parameters are conventionally used: (1) the combined volume of entrapped air and the asphalt binder *external* to the aggregate, which is referred to as the voids in the mineral aggregate (VMA) and (2) the degree to which the external binder saturates the VMA volume (voids filled with asphalt [VFA]). Both VMA and VFA have been identified as significant indicators of performance. The component diagram shown in Figure 1 is commonly used to model the mass and volumetric properties of asphalt mixtures.

Specification and application of a *minimum VMA* have been in common use since the early 1960s. Minimum specified VMA has been inextricably defined in relation to the maximum (or nominal maximum) aggregate particle size in the aggregate blend as shown in Figure 2. This research seeks to examine the premise that VMA is indeed a valid critical parameter and that the sole aggregate factor affecting the magnitude of critical VMA is the nominal maximum aggregate size.

Study Objectives

The goal of this study is to determine the validity of the minimum VMA requirement versus nominal maximum aggregate size required in Superpave volumetric mix design. The project seeks to fulfill three specific objectives:

1. to establish a laboratory method by which the transition of an asphalt paving mixture from sound to unsound behavior may be credibly identified and measured;
2. to use that method to identify and to evaluate statistically the effects of aggregate-related factors on the critical state of such mixtures; and
3. to derive a predictive relationship relating critical state (e.g., critical VMA) to aggregate-related properties such as nominal maximum aggregate size, gradation, shape, and texture.

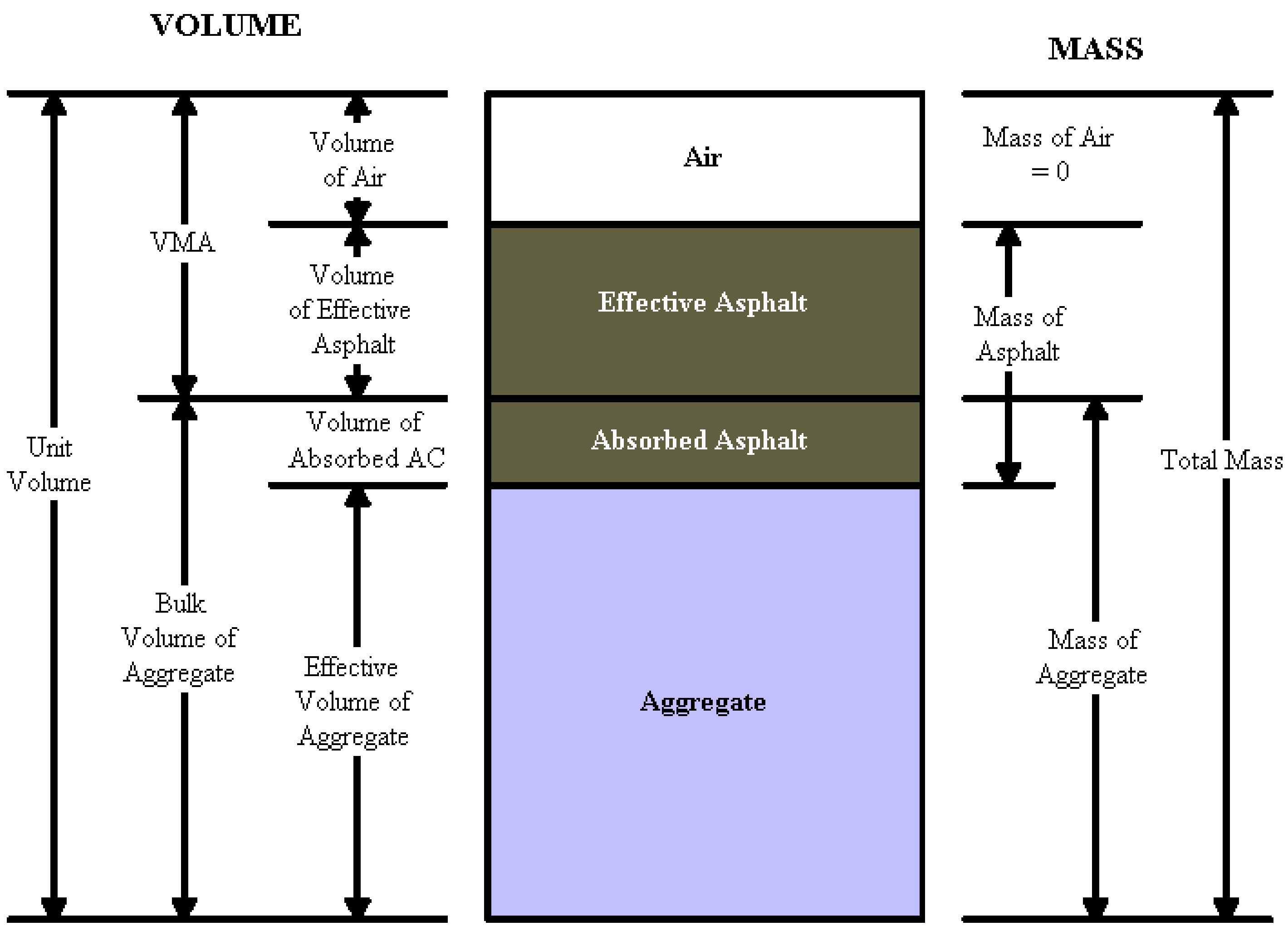

FIGURE 1 Component Diagram of Compacted Hot-Mix Asphalt Sample

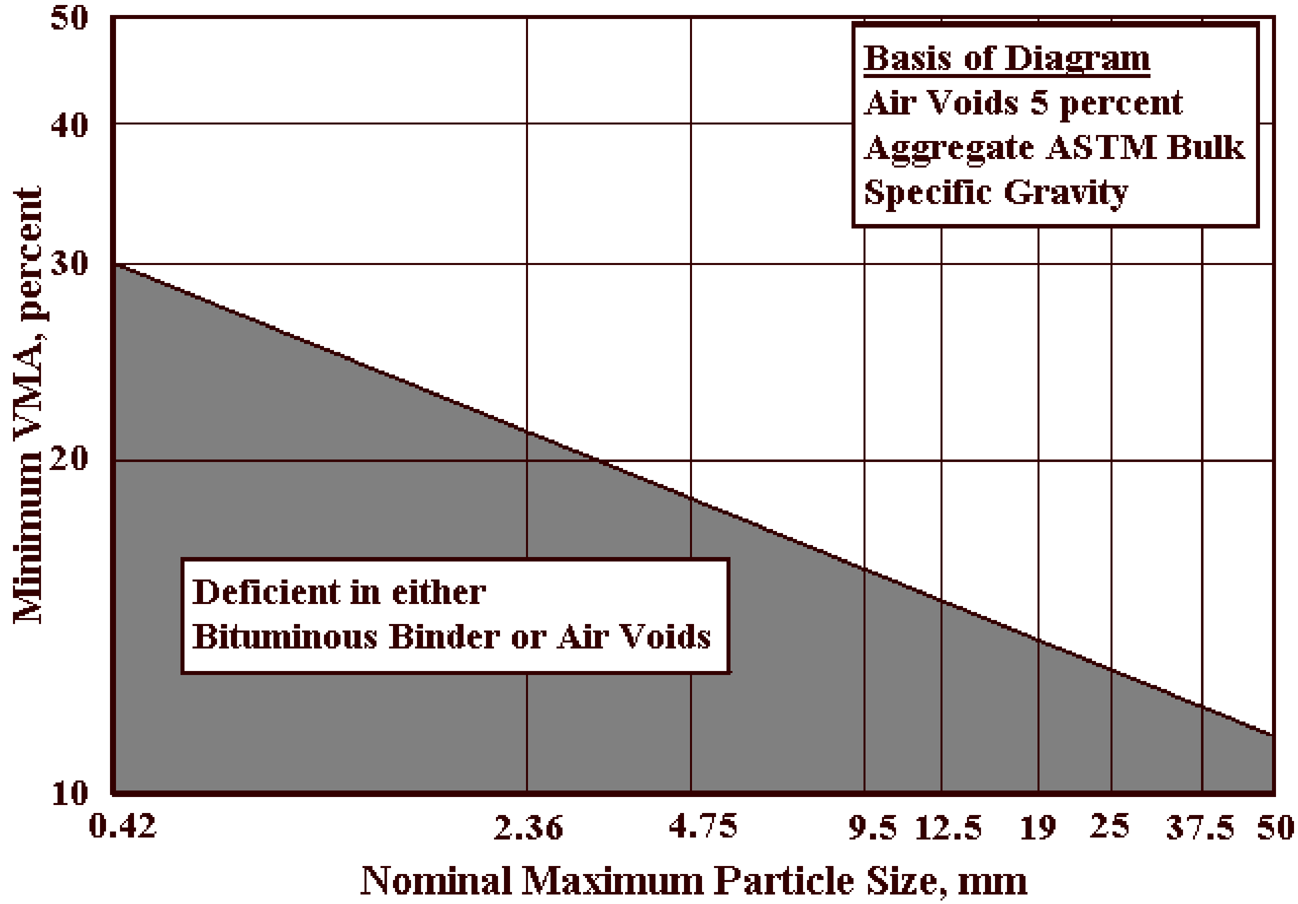

FIGURE 2 Minimum VMA Versus Nominal Maximum Aggregate Size Relationship (*2*)

Specific Tasks

To accomplish these goals and objectives, the project was broken into six tasks. The objective of task 1 of this project was stated as follows:

> A comprehensive literature review will be undertaken specifically to identify the following information:
>
> 1. laboratory methods of identifying critical state transitions in asphalt paving mixtures, including the Monismith and Vallerga method (*1*);
> 2. the history and development of the current VMA versus nominal maximum aggregate size relationship with an emphasis on validating research; and
> 3. published research results that address the effects of other aggregate-related factors on critical state transitions in asphalt paving mixtures.

A comprehensive literature search was conducted emphasizing the COMPENDEX PLUS literature database. The leading asphalt journals, e.g., those of the Association of Asphalt Paving Technologists (AAPT), the American Society for Testing Materials (ASTM), Highway Research Board (HRB), Transportation Research Board (TRB), and International Conferences on the Structural Design of Asphalt Pavements (ISAP), were also searched. The information obtained from this literature review is discussed in section 2.

The objective of Task 2 of this project was as follows:

> With the assistance of the Iowa Department of Transportation Office of Materials, identify sources of appropriate fine and coarse aggregate materials for the plan of experiment outlined in task 4; obtain sufficient amount of each aggregate for the project; and perform basic characterization testing (bulk specific gravity, absorption, gradation, shape, texture, etc.) on each to provide information for mix design and analysis.

To best accomplish this task, the research group met with members of the Iowa DOT bituminous engineering staff to identify potential aggregate sources for use in the study. Three possible sources of aggregates were located and contacted and quantities of all three materials were obtained. The aggregate materials have been sieved and tested for the relevant properties used in the study. The information obtained from this testing is presented and discussed in section 3. Also, fifty gallons of a commonly used binder, Superpave performance grade PG 58-28, was obtained. The binder information and properties are also presented in section 3.

Developing an interim report was the objective of Task 3:

> Provide an interim report summarizing the findings of tasks 1 and 2. This report will make recommendations with respect to the feasibility of performing the anticipated laboratory testing program, and provide a laboratory protocol for the work. The report will summarize the aggregates selected for the experiment and the results of the characterization testing performed on them. The report will detail the various aggregate

combinations to be used to fulfill the needs of the plan of experiment in task 4. If necessary the report will provide refined estimates of time and budget, with the justification.

The interim report was delivered to the Iowa Highway Research Board (IHRB) in October 1998.

The objective of task 4, laboratory testing, commenced upon approval of the interim report. It followed the revised plan of experiment given in Table 1. Originally it was estimated that 810 specimens would be required to be tested; this was reduced to 360 specimens because of time and material constraints and the quality and consistency of the results obtained. It is believed that the test matrix shown in Table 1 provides the essential information required to evaluate the effects of gradation, shape, surface texture, nominal maximum size, etc.

The Nottingham Asphalt Tester repeated load triaxial test apparatus was selected for use in evaluating the mixtures. This equipment allows use of the SGC-compacted specimens to examine performance of the mixture under realistic loading and temperature conditions. This type of test has been used in Europe for years; it has been used almost exclusively as a research tool, but recent improvements have made it user friendly and expedient, and it could be easily incorporated into the Superpave mix design program.

Task 5, the statistical analysis of laboratory data, involves using analysis of variance (ANOVA) and regression techniques to identify significant primary and interaction factor effects upon critical volumetric variables. Regression analysis on the identified significant factors will be used to develop an equation of the form

$$VMA_{crit} = \phi(\text{Gradation, FAA, CAA, NMAS}) + \varepsilon \quad ,$$

where FAA is fine aggregate angularity, CAA is coarse aggregate angularity, and NMAS is nominal maximum aggregate size.

This report is the objective of task 6, the final report. It is organized into six main sections including this introduction (section 1). Section 2 includes an updated summary of the literature search and review on the effects of aggregate-related factors of critical VMA in asphalt paving mixtures. Section 3 briefly summarizes the materials used in the study: asphalt and fine and coarse aggregates. Section 4 presents the laboratory method used, describing step-by-step the testing protocol used and any deviations from convention. Section 5 presents and discusses the results obtained from the testing program and statistical analysis. The significant factors are identified, and predictive equations are developed and evaluated. Conclusions and recommendations are given in section 6.

TABLE 1 Original and Revised Plan of Experiment: Experimental Matrix

Aggregate Blend		Nominal Maximum Aggregate Size and Gradation								
		9.5 mm			12.5 mm			19.0 mm		
Coarse Fraction	**Fine Fraction**	**Fine**	**Dense**	**Coarse**	**Fine**	**Dense**	**Coarse**	**Fine**	**Dense**	**Coarse**
Crushed	Natural									
	50/50									
	Manufactured	X	X	X	X	X	X	X	X	X
Gravel	Natural	X	X	X	X	X	X	X	X	X
	50/50									
	Manufactured	X	X	X	X	X	X	X	X	X
50/50	Natural									
	50/50	X	X	X	X	X	X	X	X	X
	Manufactured									

Note: X denotes the revised matrix

2 LITERATURE SEARCH RESULTS

McLeod, in proposing his minimum voids in the mineral aggregate (VMA) requirement versus nominal maximum aggregate size (NMAS) relationship did not present the research or data from which it derived and stated that “it is subject to modification as further experience and test data are *2*). In Superpave, meeting McLeod’s minimum VMA requirement is frequently the deciding factor as to whether or not an aggregate blend can be used. In recent years, some researchers have presented concerns that these minimum VMA requirements are too restrictive and may rule out economical mixes with acceptable performance properties (*3*). Others point out that evaluating and selecting the aggregate gradation to achieve a minimum VMA is the most difficult and time-consuming step in the Superpave mix design process (*4*). Others suggest it is not applicable to all asphalt mixtures and propose refinements to it (*5, 6, 7*).

Formally, as defined in task 1 of the project, there are three distinct parts to the literature search:

1. identifying laboratory methods of distinguishing critical state transitions in asphalt paving mixtures, including the Monismith and Vallerga method (*1*);
2. examining the history and development of the current VMA versus NMAS relationship with an emphasis on validating research; and
3. locating published research results that address the effects of other aggregate-related factors on critical state transitions in asphalt paving mixtures.

To accomplish these tasks, a comprehensive literature search was conducted using the COMPENDEX PLUS literature database. This database is excellent for the period from the 1970s to the present. For earlier (pre-1970s) materials, the index of proceedings of the Association of Asphalt Paving Technologists (*8*) suggested several relevant papers. Many of these papers referenced papers presented at the American Society for Testing Materials and Highway Research Board meetings and the International Conference(s) on Structural Design of Asphalt Pavements, leading to further information. The information obtained from this literature review for each of the three topics is discussed at length below.

Laboratory Methods of Distinguishing Critical State Transitions

What defines a state of critical VMA? This is not addressed by any of the conventional tests conducted in asphalt laboratories and does not show up in the literature. For the purposes of this project, the critical state transition occurs where the compacted asphalt mixture transitions from sound to unsound response to load; it becomes plastic, loses strength quickly, and begins to deform readily.

Therefore, prior to any investigation into critical VMA, a practical and credible means must be found to identify a state of critical VMA in a laboratory mixture and to identify the volumetric parameters of that mixture as it transitions from sound to unsound behavior.

The first question that needs to be addressed is What laboratory test best distinguishes the critical state transition of compacted hot-mix asphalt mixes? Since permanent deformation best describes this phenomenon, the most logical tests to consider for determining the critical state transition are those that characterize this distress. There are several approaches and test methods for examining permanent deformation.

A good starting point is the Strategic Highway Research Program (SHRP). One of the primary objectives of SHRP was to develop a series of accelerated performance-related tests. Rutting (permanent deformation) was the focus of the SHRP A-003A project and SHRP report A-415. The SHRP researchers examined a wide variety of test methods to find the best performance test for measuring permanent deformation response. While distinguishing the critical state transition was not a goal of the SHRP researchers, their review and discussion of candidate test methods is useful in identifying equipment to determine the critical transition of a mixture.

The SHRP researchers discussed four types of laboratory tests used to characterize the permanent deformation response of pavement materials (*9*):

1. Uniaxial stress tests—unconfined cylindrical specimens in creep, repeated, or dynamic loading.
2. Triaxial stress tests—confined cylindrical specimens in creep, repeated, or dynamic loading.
3. Diametral tests—cylindrical specimens in creep or repeated loading.
4. Potential (new) tests—e.g., simple shear and hollow cylinder tests.

Of these, based on field simulation and simplicity, they ranked the simple shear test (SST) first, the triaxial test second, and the creep tests third. They believed that the shear properties were the most important in rutting and that SST provided the best means for directly measuring the effects of a specific stress state and the dilation characteristics of a mix. For distinguishing the critical state transition of a compacted HMA specimen, the advantages of SST are not worth the increased cost over either the triaxial stress or the creep test apparatus.

As there is no standard test method used to identify this state transition, the Iowa State University research group decided to use NAT, which has the capability to perform triaxial testing and which has come close to being the standard testing device throughout Europe under the developing European Standards (EN). The literature review therefore focuses on this test method exclusively. The goal is to examine the existing and available literature to learn more about this test, test parameters, and the feasibility of using this equipment to distinguish the critical state transition.

Triaxial Testing

The triaxial test has been used by asphalt technologists since the early 1940s for characterizing asphalt mixtures. Most of this research was of an exploratory nature because of the cost and complexity of the test equipment. However, several influential researchers have used the test in a variety of ways.

Nijboer was one of the first to use the triaxial test for asphalt mixtures. He discussed existing test methods and rejected them as inadequate for measuring plastic properties of asphalt mixtures (*10*). He recommended against using the Hveem stabilometer because it is a “closed-system,” meaning the material cannot flow laterally. He recommended an “open-system” test in which lateral flow is possible.

The triaxial shear test, widely used in soil mechanics, is one example of such a test. Nijboer developed the triaxial test for bituminous mixtures and used it to study the influence of systematic changes in asphalt content, filler, and ratio of coarse to fine aggregate on resistance to plastic deformation.

Goetz and Chen used the vacuum triaxial apparatus with confining pressures of 14, 7, and 0 pounds per square inch (psi) and compared the results with a conventional pressure triaxial test at 25 psi (*11*). They found the vacuum triaxial apparatus to work satisfactorily with bituminous mixtures.

Monismith and Vallerga examined the relationship between density and stability using an open-system triaxial test (*1*). They used one type of asphalt (3–8 percent by weight of aggregate), one kind of aggregate and gradation, and a test temperature of 60 degrees C. They molded specimens using several different compaction schemes (pressure and tamping). Then they ran triaxial compression tests, using a lateral pressure of one to two bar and applying the vertical load at a constant rate of strain of 0.5 inches per minute.

Their test results suggest that during compaction HMA behaves analogously to a cohesive soil in proctor testing. They found that the relationship between density and stability depends on how stability is defined. They found that existing methods for identifying stability allowed considerable variability in the magnitude of strain at which stability is determined. Figure 3 shows the relationship between bulk specific gravity and stress for mixtures with binder contents between three and seven percent at two percent strain. The figure shows that for binder contents above five percent, there is a maximum density beyond which the specimen begins to lose strength. The dashed line is the Hveem design binder content (5.6 percent by weight of aggregate) with compaction achieved by construction and one year of traffic. As shown in the figure, after this time the mix would have a significant loss of stability.

Pell and Brown stressed the importance of reproducing in situ test conditions in laboratory tests and critically reviewed existing test methods (*12*). They suggested that the repeated load triaxial (RLT) test will overestimate the permanent deformation characteristics of a mix relative to in situ conditions. They emphasized the need for direct shear testing to supplement repeated load triaxial testing for pavement design.

Morris, Haas, and Reilly suggested that there is an interaction between confining stress and temperature (*13*). The effect of the confining stress becomes important at higher temperatures.

Francken used a repeated load triaxial apparatus to determine a phenomenological deformation law that could then be used in structural design to limit rutting (*14*). Examining five different mixes, he found that a threshold condition (dependent on stress and temperature) existed that clearly delineated whether or not plastic failure was imminent.

Brown and Cooper examined a variety of mixes for bases and base courses using several tests, including Marshall stability, uniaxial and triaxial creep, and the repeated load triaxial test (*15*). They concluded that the Marshall stability test could not be used to distinguish the relative deformation resistances of these mixes and stated that “if a confined test is to be used, it is necessary to apply some *15*).

The SHRP researchers (*9*) pointed out that previous research had suggested that the repeated load test was more sensitive to mix variables than the creep test. They found that the repeated load triaxial test provided a better measure of rutting characteristics than the creep test.

Brown and Gibb compared the RLT apparatus with wheel tracking in the pavement test facility (PTF) at the University of Nottingham (*16*). They compared the performance of two mixes (one gravel, one granite) in the PTF with cored samples (same mixes) in an RLT apparatus. They tested at 40 degrees C and a confining pressure of 70 kPa. Their results showed that confinement “improved” the gravel mix such that it compared favorably with the granite mix, whereas in the PTF there were pronounced differences. This led them to remark that the “sensitivity of gravel aggregate mixtures to test conditions suggests that some care will be needed in specifying mixture design tests” (*16*).

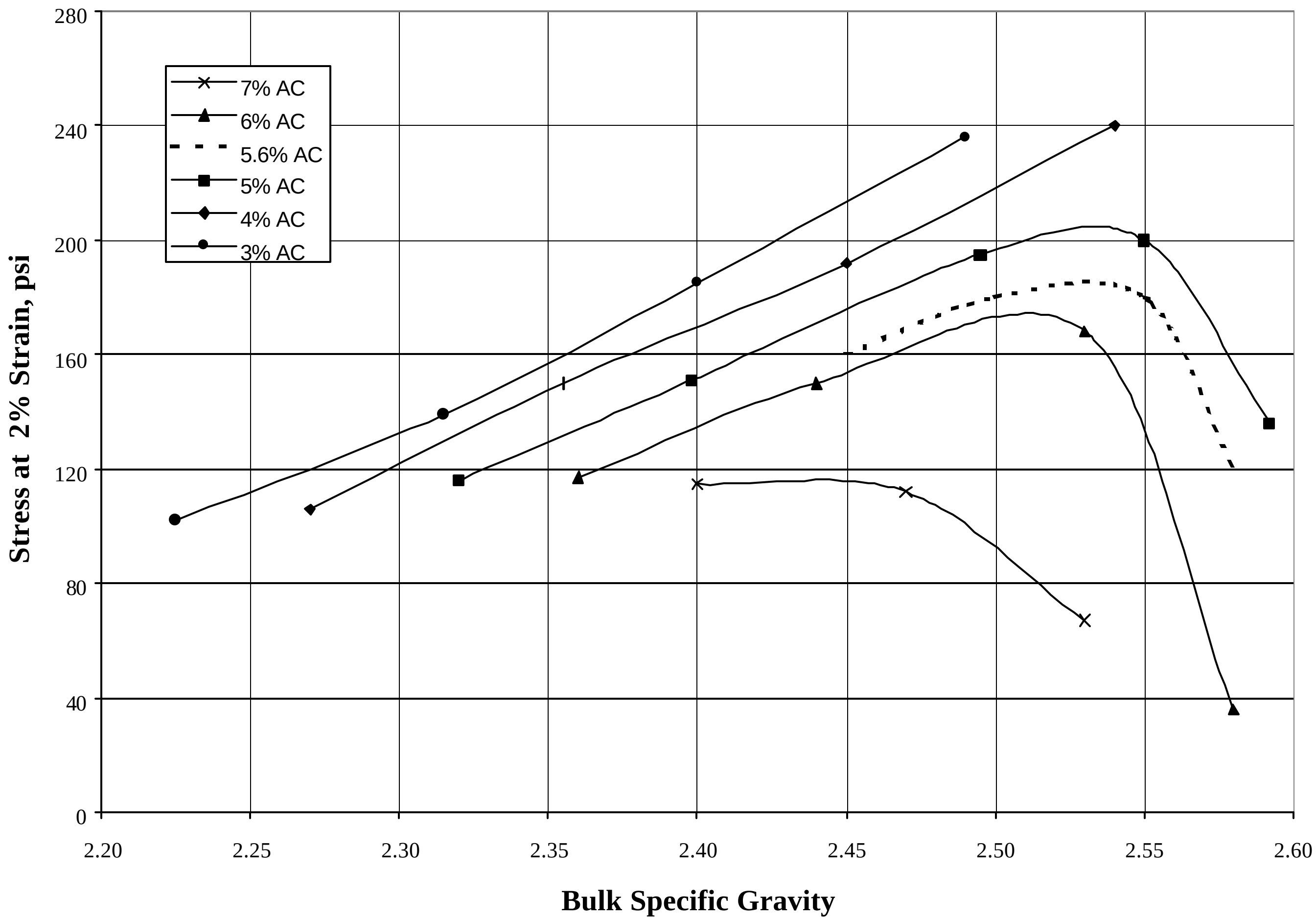

FIGURE 3 Stress at Two Percent Strain Versus Bulk Specific Gravity (*1*)

Nunn, Brown, and Guise compared the repeated load axial test (both confined and unconfined) with wheel-tracking tests of the same materials and found that the RLT test ranked the materials in a similar fashion to the wheel-tracking test (*17*). They found the unconfined test inadequate for evaluating resistance to permanent deformation. They recommended that the RLT test be further evaluated to develop standard test conditions and a precision statement.

Brown and Scholz also modified NAT to convert the repeated load axial test into a repeated load triaxial test, using a vacuum to apply the confining stress (*18*). This approach limited the confining stress to one atmosphere (roughly 100 kPa) but made the test viable as a routine test. They then used the apparatus to examine two porous mixtures with the same gradation but different binders at different temperatures and confining stresses. They found that confining the specimen emphasized the role of aggregates in resisting permanent deformation.

The History and Development of the Current VMA Versus Nominal Maximum Aggregate Size Relationship

In the early 1900s, the most widely used approaches to asphalt mix design focused on achieving maximum density or using surface area and film thickness to determine the optimum asphalt content (*19*). Mix designers in the first group combined VMA, air voids, and experience to determine the best asphalt content. Those using the second approach combined air voids, the product of surface area and optimum film thickness, and experience to determine the best asphalt content. The Hubbard-Field design method is an example of the first approach, and the Hveem design method an example of the second. Because experience was usually the critical factor, regardless of approach, they usually resulted in similar mix designs. Usually, the aggregate gradation was determined by specification, by locally available materials, or by theoretically "idealized" gradations.

The "early" Marshall mix design approach did not have a VMA requirement. Marshall himself believed "no limits can be established for VMA, for universal application, because of the versatile application of bituminous materials to many types and gradations of aggregate" (*20*). McFadden and Ricketts presented the Corps of Engineers (COE) version of the Marshall method for design and field control of paving, which used the five parameters shown in Table 2 to determine the design asphalt content (*21*). The peak values of all parameters except flow were averaged to determine the design asphalt content.

TABLE 2 Corps of Engineers Marshall Mix Design Criteria (*21*)

Test Property	Requirement
Stability	500 pounds (minimum)
Flow	20 (maximum)
Air voids, total mix	3–5 percent
VFA	75–85 percent
Unit weight	—

The shift towards a minimum VMA requirement began in the mid-1950s. McLeod in 1955 presented his initial analysis on "the voids properties of compacted paving mixtures," in which he laid out the basic principles of a minimum VMA requirement (*22*). His argument did not explicitly mention durability; he was concerned that specifications with requirements on both air voids and VFA were too restrictive at higher asphalt contents. He showed for absorptive aggregates that computed VMA and VFA would be wrong unless the bulk specific gravity was used in the calculations.

In 1956, McLeod presented a modified Marshall mix design methodology, which listed a minimum VMA requirement of 15 percent (*23*). He showed graphically (see Figure 4) that a VFA range of 65-80 percent was unachievable for mixes with asphalt contents above 10.5 percent by weight (approximately 20 percent by volume). He provided similar design charts that covered the range of aggregate specific gravity from 2.00 up to 3.00 and asphalt specific gravity from 0.95 up to 1.11; in all cases the minimum asphalt content required would be at least four percent by aggregate weight, plus any absorbed asphalt. At a typical aggregate specific gravity G_{sb} = 2.65 and asphalt specific gravity of 1.01, McLeod's design charts specify a minimum asphalt content of 4.5 percent. McLeod believed that the physical test limits would broaden the range of acceptable aggregates, lower the cost of bituminous paving mixtures, and provide satisfactory paving mixtures with respect to stability, voids, durability, etc.

The following year McLeod again stated his case for using the bulk specific gravity and effective asphalt content for volumetric analysis of the mixture (*24*). He concluded that if the compacted paving mixture was restricted to three to five percent air voids, requiring a minimum VMA (15 percent) was less restrictive than requiring a VFA range of 75 to 85 percent. More important, he suggested that the VFA requirement would allow a pavement to be constructed with 3.76 percent asphalt, which he felt was too low for durability. The minimum VMA requirement would ensure at least 4.5 percent asphalt and provide adequate durability. McLeod observed that Canadian aggregates typically were too densely graded to provide the required VMA.

Also in 1957, Lefebvre reemphasized the importance of minimum VMA (*25*). Aware of the difficulty of achieving 15 percent voids in the mineral aggregate and three to five percent air voids, he investigated the influence of the principal fractions of the mineral aggregate—coarse aggregate, fine aggregate, fine sand, and mineral filler—on the performance of the paving mixture. He found that the fine aggregates were the most critical component, controlling the VMA and contributing to stability.

In 1959 McLeod suggested the currently used method of using VMA and air voids requirements in designing pavement mixtures (*2*). In place of his previously held requirements of 15 percent minimum VMA, he related minimum VMA to nominal maximum particle size. Figure 2 shows McLeod's suggested relationship. He warned that the minimum VMA requirements were subject to modification as further experience and additional test data were accumulated.

In 1959 Campen et al. emphasized that asphalt film thickness, not VMA, was essential to mixture durability (*26*). VMA is independent of the surface area of the aggregate. They presented data showing that two aggregate blends could have identical VMA and one could have twice the surface area and film thickness as the other. At the same time, they found that the surface area did not indicate the asphalt content required for minimum VMA. Increased surface area requires more asphalt, but there is no direct proportional relationship. They prescribed film thicknesses in the range of six to eight microns as producing the most desirable paving mixtures.

The Asphalt Institute incorporated a new density-voids analysis, which accounted for asphalt absorption, into the Marshall mix design method, in its 1962 MS-2 (*27*). VFA, previously a Marshall method design parameter in earlier editions, is not mentioned. No rationale for dropping VFA is presented. McLeod wrote an appendix in MS-2 presenting the inclusion of a minimum VMA requirement into the mix design process.

Hudson and Davis described an arithmetical method for computing VMA from the aggregate gradation (*19*). Using factors for the ratio of percent passing one sieve divided by the percent passing the next smaller sieve. Their procedure differentiated between rounded and angular aggregate. They believed that their arithmetic method of computing VMA would allow the mix designer to estimate design asphalt content if McLeod's chart (Figure 2) was used.

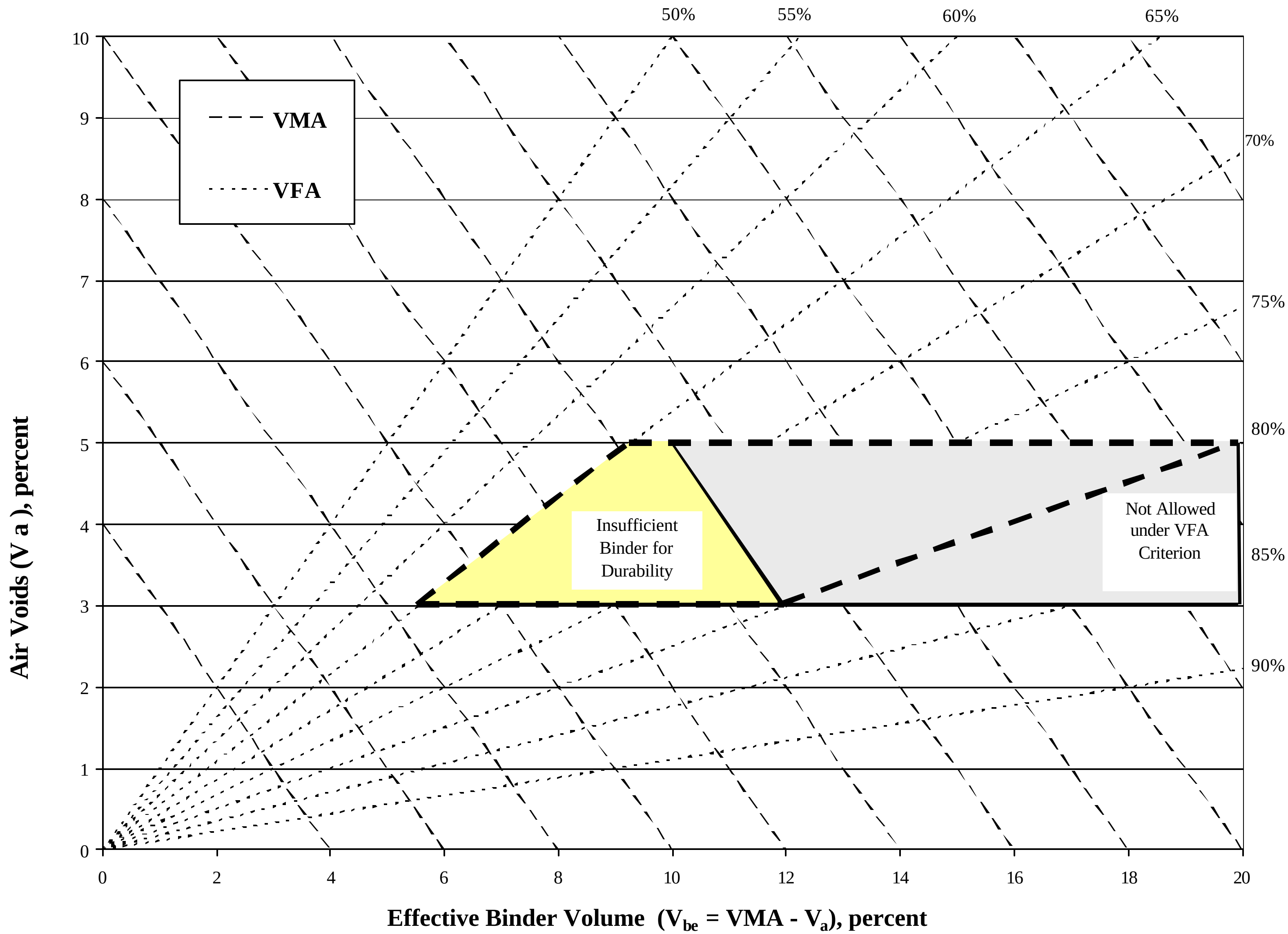

FIGURE 4 McLeod's Concerns with VMA Criterion

McLeod discussed the trend of modifying paving mixtures with rubber or asbestos to increase durability (*28*). As an alternative, to improve durability, he proposed using a conventional (unmodified) asphalt binder but requiring two to three percent more VMA than the values shown in Figure 2. He demonstrated that the VMA value of a dense graded paving mixture essentially controls the quantity of asphalt that can be incorporated into the mixture. Also, he argued that VMA should be determined through measurements of compacted mixtures; it cannot be determined from aggregate test properties alone. He offered several methods to increase VMA—most important, using crushed angular aggregates.

Field presented the results of a study investigating the minimum VMA criterion and the accuracy of the test and examining alternative approaches (*29*). He pointed out that the Ontario Ministry of Transportation and Communications (MTC) had supplied acceptable mixes that did not meet the required minimum VMA. The MTC was changing its requirements to those shown in Table 3, where it must be noted that the maximum size is the same as the Superpave nominal maximum size.

TABLE 3 Ontario Ministry of Transportation and Communications Modification to VMA Requirements (*29*)

Mix Type	Percent Pass 4.75 mm* (by mass)	Nominal Maximum Particle Size (mm)						
		2.36	4.75	9.5	13.2	16.0	19.0	26.5
HL-2		21	18.0	16				
HL-1	40				13.5	13.0	12.5	11.5
HL-3	45				14.0	13.5	13.0	12.0
HL-4	50				14.5	14.0	13.5	12.5
HL-5	55				15.0	14.5	14.0	13.0
HL-6	60				15.5	15.0	14.5	13.5
HL-8	65				16.0	15.5	15.0	14.0

*When the difference between the bulk relative density of the retained 4.75-millimeter material and the bulk specific gravity of the pass 4.75-millimeter material is greater than 0.3, then the percent pass 4.75 millimeters must be on a volume basis.

Notes: The VMA shown above is for 3.5 percent voids. Reduce the VMA shown above by amount of voids set less than 3.5 percent. Increase the VMA shown above by amount of voids set more than 3.5 percent. A design mix must have at least a moderate to moderately rich asphalt coating appearance on aggregate particles before compaction.

Field also discussed four alternative approaches to using minimum VMA in getting mix durability:

1. a VFA requirement,
2. the surface area method,
3. the centrifuge kerosene equivalent (CKE) test, and
4. visual observation of coat-ability.

A VFA requirement of 75–85 percent was ruled out because it would allow mixes with very low VMA and very low asphalt contents to be used. The surface area method provided mixes with average design asphalt contents 1.2 percent lower than those obtained using the VMA criterion. So, despite good laboratory test properties (excepting low VMA!) and no construction or performance problems, because of conceptual problems the method was deemed unacceptable. The CKE approach was found unsatisfactory because it is "lengthy, tedious, subject to many errors, and not realistic" (*29*). Using visual observation for coat-ability was deemed acceptable based on past projects where it had been used. The criteria involved making sure (1) the loose mix was moderately rich with respect to asphalt, (2) the compacted test specimen was moderately rich to rich in appearance, and (3) the aggregate particles were well coated with asphalt. Field concluded that the minimum VMA requirement based on bulk specific gravity was the best method of establishing proper asphalt content for durability. Field also recommended follow-up performance studies be conducted on pavements with VMA and void contents below the design criteria to provide the necessary experience and confidence.

Kandhal and Koehler reported there were still problems with the VMA criterion in 1986 (*30*):

> The VMA is considered to be the most important mix design parameter which affects the durability of the asphaltic concrete mix. High VMA values allow enough asphalt to be incorporated into the mix to obtain maximum durability without the mix flushing. Additionally, such mixes have the following advantages compared to low VMA mixes:
>
> 1. Lower stiffness modulus at low temperatures. This is helpful in minimizing the severity of thermal and reflection cracking.
>
> 2. Lower susceptibility to variations in asphalt and fines content during production. Such variations can cause the mix to be too brittle or too rich.
>
> Unfortunately, only 16 of 38 states using the Marshall method specify a minimum VMA. Of these 16 states, only seven use the effective asphalt content (total asphalt minus the asphalt absorbed by the aggregate) to calculate the realistic VMA value, as recommended by the Asphalt Institute. If the effective asphalt content is not used, the calculated VMA values are not reliable especially when the mix contains an absorptive aggregate.

Foster reviewed the use of voids in mix design and specifications (*31*). While acknowledging McLeod's explanation of VMA as providing "the desirable conditions for a good asphalt pavement," he questioned the minimum requirement of 15 percent VMA. He reviewed McLeod's 1956, 1957, and 1959 papers and Lefebvre's 1957 paper and pointed out that none report actual pavement VMA or performance data in support of the recommended criteria. Foster reported that as of 1985 seventeen states were using VMA in their mix designs. He compared pavement performance data from several projects, and his data are presented graphically in Figures 5 and 6.

Figure 5 presents graphically the volumetric mix data from traffic tests that the United States Army Corps of Engineers used to develop their Marshall design criteria. The nominal maximum size was (primarily) 19.0 millimeters (0.75 inches). The data clearly show the importance of the three to five percent air voids criterion. For VFA, a criterion of 68 to 77 percent (approximately) will result in satisfactory pavements. The VMA criterion shows that a minimum of 14 percent is necessary to distinguish the "almost plastic" pavements but does not break out the "almost brittle" pavements.

Figure 6 presents graphically the volumetric mix data from 18 experimental overlays on Nebraska highways from 1961 to 1972. The rings differentiate the different mix types; nominal maximum size was (primarily) 19.0 millimeters (0.75 inches). The data clearly show that a VFA criterion of 68 to 83 percent (approximately) will result in fair or good pavements. The VMA criterion is ineffective at distinguishing pavement performance in this data. Interesting to note,

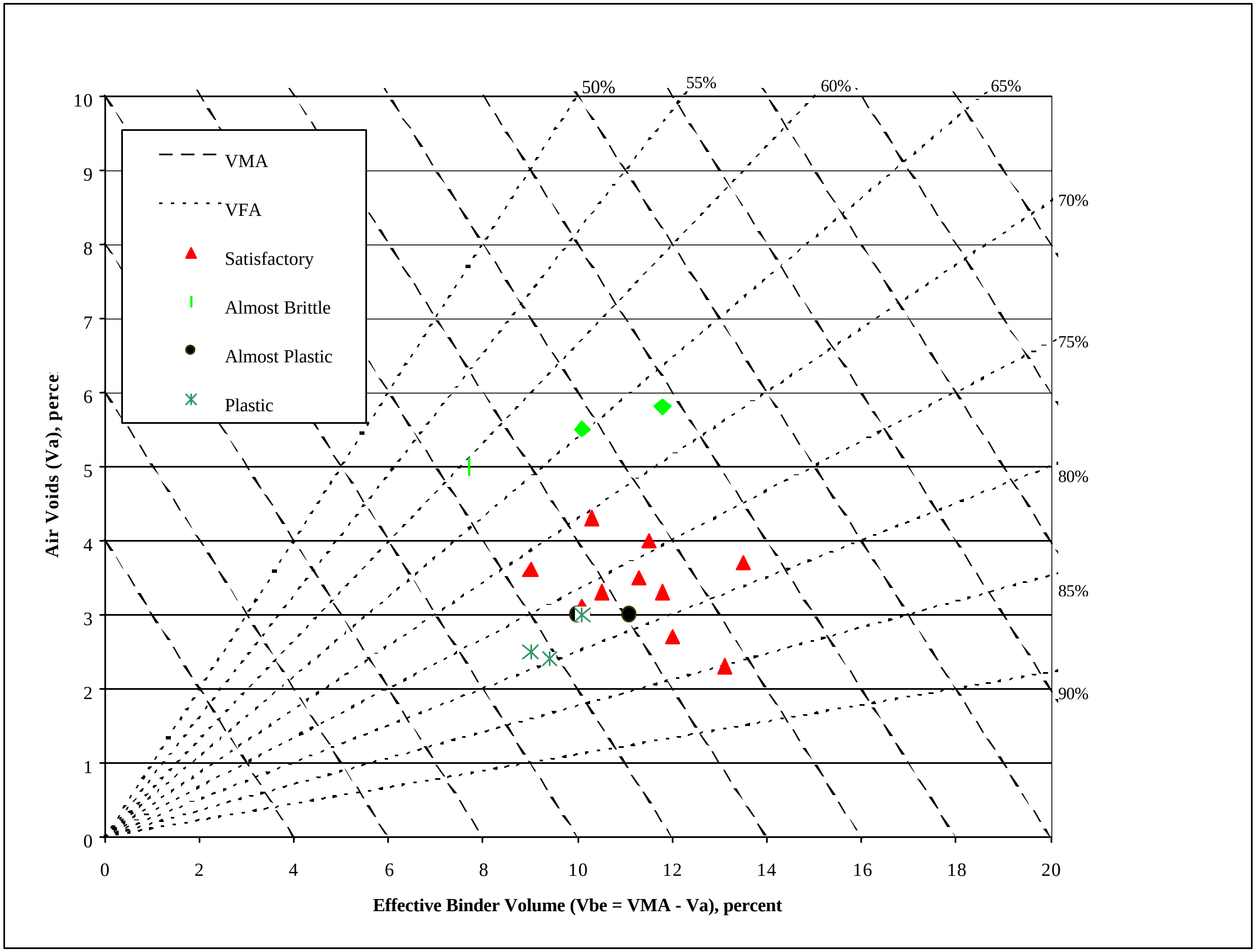

FIGURE 5 Ineffectiveness of VMA to Distinguish Pavement Performance (*31*)

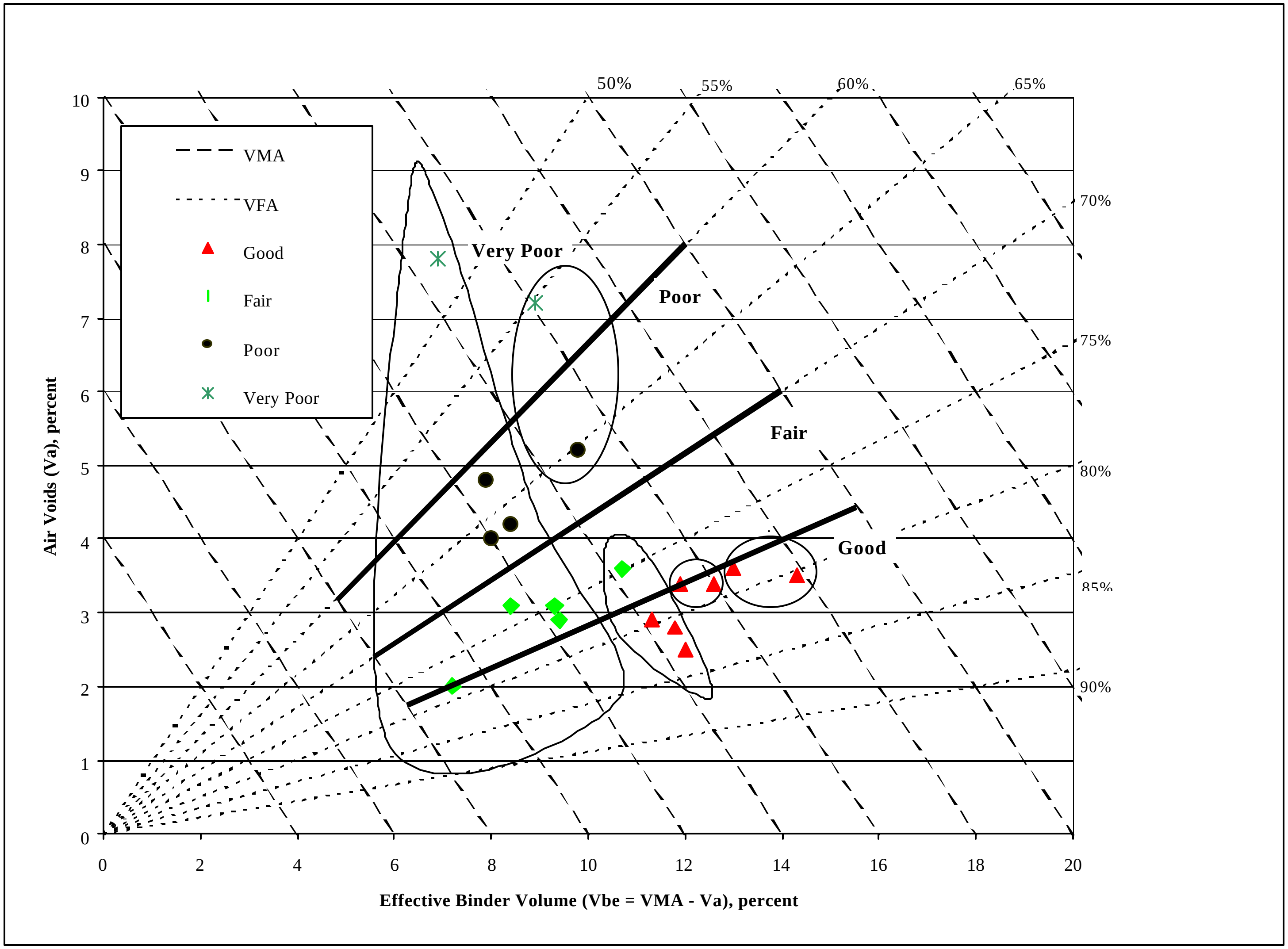

FIGURE 6 Effectiveness of VFA for Predicting Pavement Performance (*31*)

Foster also had film thickness information for these projects that also did not correlate well with performance.

Huber and Heiman examined nine test sites in Saskatchewan to see whether mix design characteristics differentiated pavements that performed well from those that rutted badly (*32*). For the mix characteristics examined, they found the threshold values listed in Table 4. If four percent air voids are taken as a design target, their VMA and VFA criteria limit possible designs to a single point (air voids = four percent, VMA = 13.5 percent, and VFA = 70 percent). They concluded that asphalt content and voids filled with asphalt were the most basic parameters that affect rutting, with VFA including the effects of both air voids and VMA.

TABLE 4 Observed Threshold Values for Mix Design Characteristics (*32*)

Parameter	Threshold Value
Air voids	4 percent minimum
Voids in the mineral aggregate	13.5 percent minimum
Asphalt content	5.1 percent maximum
Voids filled with asphalt	70 percent maximum
Fractured faces	60 percent minimum
Marshall stability	—
Hveem stability	37 percent minimum

McLeod reemphasized his earlier arguments for using VMA in mix design (*33*). Aware of Huber and Heiman's findings (*32*), he acknowledged that there was apparent justification for using air voids and VFA as design criteria. However, using an air voids and VFA criteria of 75–85 percent would not be a practical specification for production. He further argues against placing requirements on all three volumetric parameters (air voids, VMA, and VFA), showing that they overlap. As a practical matter, he suggests, the only reasonable criteria is to use the minimum VMA based on nominal maximum particle size and an air voids requirement. He mentions that in Ontario during the OPEC oil crisis of 1973, the VMA requirements were significantly reduced as a cost-saving measure, but the reductions quickly halted due to an epidemic of poor pavements and raveling problems.

Huber and Shuler focused on the relationship between VMA and the maximum density line (MDL; *34*). They concluded that the MDL needed to run from the origin to the 100 percent passing maximum sieve size. They tried to relate distance from the MDL to VMA but could find no general rule to ensure minimum VMA because of the influence of aggregate angularity and surface texture on VMA. They also recommended against comparing gradations with large differences in material passing the No. 200 sieve.

Cominsky, Leahy, and Harrigan presented and discussed the Superpave level 1 mix design that was developed during the Strategic Highway Research Program (*35*). Based on the recommendations of a panel of experts using the Delphi method, the VMA requirements were absorbed into Superpave. The panel's final rating of the various aggregate and asphalt-aggregate mixture characteristics for inclusion into the specification is shown in Table 5. As can be seen, the panel strongly recommended air voids and VMA but was essentially neutral on VFA, dust-asphalt ratio, and film thickness.

In 1994, the Asphalt Institute reintroduced a VFA criterion into Marshall mix design, changed the design air voids to four percent, and added a table of VMA requirements depending on air voids and nominal maximum aggregate size (*36*). The stated purpose of the VFA criterion was to limit the maximum values of VMA and asphalt content.

TABLE 5 Average Ratings of Asphalt-Aggregate Mix Characteristics by SHRP Expert Task Group (*35*)

	Rating*	Standard Deviation	"Best" Measurement
Air voids	6.77	0.44	Rice specific gravity
VMA	6.15	0.90	Bulk specific gravity of aggregate (G_{sb})
VFA	4.00	1.68	None identified
Dust-asphalt ratio	4.46	1.85	None identified
Film thickness	3.31	1.89	MS-2 procedure

*Scaled rating: 1, very strongly disagree; 2, strongly disagree; 3, disagree; 4, neutral; 5, agree; 6, strongly agree; 7, very strongly agree.

Aschenbrenner and MacKean examined 101 mix designs to determine which MDL worked best for predicting VMA, achieving the best correlation with the Superpave definition (*37*). They reported that in 1993, the first year the Colorado Department of Transportation specified a minimum VMA, the average mix design asphalt content increased by 0.46 percent.

Kandhal and Chakraborty set out to reexamine the rationale behind the minimum VMA requirements currently being used and to establish an optimum film thickness for mix durability (*5*). Like Foster, they could not find any significant rational data correlating pavement performance with the currently specified minimum VMA values for HMA mix design. They tested mixtures with six effective asphalt film thicknesses, aged both short and long term, and they tested specimens for resilient modulus and tensile strength. They also tested the recovered binder for penetration, viscosity, complex modulus, and phase angle. In their studies they found that asphalt film thickness correlated well with resilient modulus, and they recommended an average film thickness of 9–10 microns for specimens compacted at eight percent air voids. Interesting enough, a nine micron film thickness at four percent air voids would require a minimum VMA of 15.6 percent, 1.6 percent higher than the Superpave specification.

Hinrichsen and Heggen also proposed using average film thickness in mix design (*3*). They provided equations that used the aggregate gradation and volumetric properties to determine the proper VMA for each mix design uniquely. To do this, they took the standard film thickness equation, assumed a standard film thickness, and back-calculated the amount of asphalt required providing this film thickness. Using volumetric relations, they computed the minimum VMA allowable with this asphalt content and a target air voids. They provided information that showed that mixes based on minimum VMA were not always the best in terms of performance and economics. They questioned the use of "rigid" minimum VMA specifications, showing that there is considerable variability in the tests performed to determine VMA, resulting in a standard deviation of 1.3 percent for VMA.

Anderson and Bahia found achieving VMA the most difficult and time consuming step in Superpave volumetric mix design (*4*). They analyzed 128 trial gradations from 32 mix designs performed by the Asphalt Institute from 1992 to 1996 to determine whether they could make any recommendations toward selecting an aggregate gradation. Their analysis agreed with prior researchers that VMA is dependent on more than just aggregate gradation. They found that current methods for increasing VMA were not absolutely effective. Their best recommendation to meet VMA requirements was to develop an S-shaped gradation curve ($r^2 = 0.58$) or to use the sum of the distances from the MDL ($r^2 < 0.20$).

Kandhal, Foo, and Mallick assumed asphalt mix durability was dependent on film thickness *(6*). Based on average film thickness, they found the current minimum VMA requirements inadequate for ensuring mix durability. They concluded that it penalized coarse graded mixes with low VMA but with adequate film thickness. They recommended dropping the minimum VMA requirement in place of a minimum average film thickness of eight microns. While they could not find the background research data on which the Asphalt Institute surface area factors are based, they felt they should still be used.

Mallick et al. point out that McLeod used relatively fine-graded mixtures to develop his relationship (*7*). Examining 9.5-, 12.5-, 19.0-, 25.0-, and 37.5-millimeter NMAS mixes, they found on average that a five percent increase in percent passing the 2.36-millimeter sieve would increase the VMA by 0.4 percent. They suggested that a more rational way of specifying VMA would be to specify VMA by the percent passing the 2.36-millimeter sieve. Their recommended design VMA requirements for dense-graded mixes are presented in Table 6.

TABLE 6 Proposed Minimum VMA based on NMAS and P2.36 Millimeters (*7*)

9.5 mm		12.5 mm		19 mm		25 mm		37.5 mm	
P2.36*	**VMA**	**P2.36***	**VMA**	**P2.36***	**VMA**	**P2.36***	**VMA**	**P2.36***	**VMA**
67–62	16.6	58–53	15.8	49–44	14.0	45–40	13.8	41–36	13.6
62–57	16.2	53–48	15.5	44–39	13.7	40–35	13.4	36–31	13.2
57–52	15.7	48–43	15.2	39–34	13.4	35–30	13.1	31–26	12.8
52–47	15.4	43–38	14.9	34–29	13.1	30–25	12.7	26–21	12.2
47–42	15.0	38–33	14.5	29–23	12.7	25–19	12.3	21–15	11.7
42–37	14.6	33–28	14.1						
37–32	14.2								

Effects of Other Aggregate-related Factors on Critical State Transitions

In McLeod's 1957 paper he summarized the principal factors influencing VMA as follows (*24*):

1. For any given particle size, the Fuller or Weymouth curve should produce maximum density.
2. Moving off the maximum density curve (to either side!) should provide less density and more VMA.

3. Using slightly more (or less) fine aggregate than that of the maximum density curve should open space between the coarser particles resulting in higher VMA.
4. Using appreciably less fine aggregate will result in an "open graded" mixture with relatively high VMA.
5. If the quantity of fine material ranges from slightly less to appreciably more than the Fuller curve, the VMA in the resulting dense graded mixture will increase steadily (slowly) but so will the required asphalt content such that the air voids will still be in the range of three to five percent.
6. Choosing to add or reduce fine aggregate depends on (1) required pavement surface texture, (2) whether or not the resulting pavement would be durable enough for local climate and traffic conditions, and (3) the relative cost of coarse and fine aggregates.
7. Mineral filler can rapidly increase VMA.

Lefebvre investigated the influence of the principal fractions of the mineral aggregate—coarse aggregate, fine aggregate, fine sand, and mineral filler—on the performance of the paving mixture (*25*). He found that the fine aggregates were the most critical component, controlling the VMA and contributing to stability. His recommendations included using a moderately high percentage of fine aggregate containing a small percentage of fine sand. The fine aggregate should be angular, with rough surface texture, and suitably graded. The coarse aggregates, while good for stability, are bad for VMA particularly if mineral filler is present. Mineral filler was not recommended because it fills voids and takes the place of bitumen and may be detrimental to durability.

Vallerga examined how aggregate characteristics of size, shape, and surface roughness affect the stability of asphalt paving mixtures (*38*). Based on triaxial testing, he concluded that the most important aggregate characteristic was surface roughness and believed that size and shape were less important than generally believed.

Campen et al. stressed that a satisfactory mixture is one where the aggregate contains enough voids to permit the addition of sufficient asphalt to provide comparatively thick films without filling all the voids in the aggregate (*26*). They showed data suggesting that engineers typically use a high–coarse aggregate content to control the voids.

Hudson and Davis felt VMA depended on the following conditions (*19*):

1. particle arrangement or degree of compaction;
2. the relationship between sizes of aggregate particles, in particular, the ratio between percents passing adjacent sieves; and
3. the range of size between fine and coarse materials and aggregate shape.

Field discussed how the Ontario MTC adjusted the Asphalt Institute's standard VMA requirements (*29*):

- For aggregates near the borderline acceptable VMA, if the percent passing No. 4 sieve was increased by five percent, the required VMA increased by 0.5 percent.
- For aggregates of good VMA with desirable mix characteristics—cohesion, stability, and coat-ability—if the passing No. 4 sieve was increased by five percent, the required VMA increased by 0.8 percent.
- The minimum VMA should correspond to a minimum air voids content; e.g., if VMA of 15 percent is required for air voids of 5 percent, then if design air voids are decreased, the minimum VMA should decrease correspondingly.

Aschenbrenner and MacKean examined 24 laboratory mixes to study the effects of four variables on VMA (*37*):

1. gradation,
2. percent passing 75-micron sieve (filler),
3. size distribution passing 75-micron sieve, and
4. fine aggregate angularity.

They found that gradation played a role in influencing VMA but had such poor correlation that VMA could not effectively be predicted from gradation. The percent filler significantly affects VMA, in particular, for gradations on the fine side of the MDL. Lower percent passing 75-micron sieve increased VMA; higher reduced VMA. They recommended that the fine aggregate be kept well off the MDL. Their results examining size distribution passing the 75-micron sieve were inconclusive. They found aggregate angularity to substantially affect the VMA, with crushed aggregates providing more VMA and rounded aggregates less. The fine aggregate angularity was more influential for coarse mixes or mixes following the MDL than for mixes on the fine side of the MDL.

Epps and Hand examined Superpave mixes for mixture sensitivity to asphalt content and percent passing the 75-micron sieve and found the coarse mixtures to be extremely sensitive to small changes in both (*39*). They listed the following aggregate-related factors as contributing to mixture sensitivity (*39*):

1. rounded or subrounded aggregates,
2. aggregates with smooth surface texture,
3. an aggregate blend with a high fine aggregate fraction,
4. an aggregate blend with a high natural sand content, and
5. aggregate blends with a high to intermediate sand content.

Summary

The purpose of this literature review is threefold: (1) to examine available laboratory tests for determining the critical transition from sound to unsound mixture, (2) to review how the minimum VMA criterion currently specified in Superpave developed (and any proposed refinements), and (3) to locate any information on other aggregate-related factors, e.g., gradation, particle shape, or texture.

There is general agreement that the laboratory tests best suited for determining the critical state transition are the permanent deformation tests. Reviewing the literature, there is not a consensus as to which laboratory test would best distinguish the critical state of VMA. Based on cost, availability, ease of use, and the SHRP findings (*9*) the repeated load triaxial test apparatus was the selected method.

The available literature on the development of the minimum VMA criterion is sketchy; McLeod presented his relationship without the research or data from which it derived. He anticipated that it would be modified with experience and test data; the implementation of Superpave has renewed focus on how the minimum VMA requirements impact mix design. Several researchers have pointed out and discussed problems with the VMA criterion in Superpave volumetric mix design, and a few have proposed changes (*3, 4, 5, 6, 7, 29*). These changes have centered on modifying the minimum VMA criterion to differentiate coarse and fine gradations. A few have argued for replacing the minimum VMA versus nominal maximum aggregate size criterion with a minimum asphalt film thickness specification.

Several researchers have pointed out aggregate factors other than nominal maximum aggregate size that affect VMA. These include percent filler, shape, surface texture, percent crushed aggregate, fine aggregate angularity, and coarseness of the gradation.

The aggregate factors that seem most important are surface texture, shape, and gradation. Of these, gradation is obtained by performing a sieve analysis, but surface texture and shape are not so easy to measure.

3 MATERIAL PROPERTIES

Selection of materials was undertaken with the assistance of the Iowa DOT bituminous materials engineer and his staff. It was decided that the asphalt used for the study would be a grade commonly used in Iowa, an unmodified Superpave performance grade PG 58-28 binder. Selecting the aggregates involved considerably more work, as the goal was to find local sources of both manufactured and natural aggregates, to obtain sufficient quantities of each for the project and to characterize the materials using basic tests, e.g., specific gravity, absorption, gradation, shape, and texture.

Asphalt Binder

As the binder was not intended to be a variable in the study, it was important that it be of a typical performance grade specified for use in Iowa. Jebro, Inc., of Sioux City, Iowa, supplied 10 five-gallon pails of a conventional (i.e., unmodified) PG 58-28 binder. The binder test results and American Association of State Highway and Transportation Officials (AASHTO) MP1 specification requirements are listed in Table 7.

TABLE 7 Superpave Test Properties of Asphalt Binder Used in Laboratory Testing

Test	Measured Test Results	Specification Requirement
	Unaged Properties	
Rotational viscosity at 135 degrees C	0.247	3.0 maximum
Dynamic shear at 10 rad/s kPa	1.024 at 58 degrees C	1.0 minimum
	Rolling Thin Film Oven (RTFO) Residue	
Mass loss (percent)	0.248	1.0 maximum
Dynamic shear at 10 rad/s kPa	2.515 at 58 degrees C	2.2 minimum
	Pressure Aging Vessel (PAV) Residue	
Dynamic shear at 10 rad/s kPa	4253 at 19 degrees C	5000 maximum
Creep stiffness at 60 s, MPa	239 at –18 degrees C	300 maximum
m-value	0.303 at –18 degrees C	0.300 minimum

Aggregates

Because the focus of the study was how aggregate-related factors affect critical VMA it was essential to select aggregates that were measurably different using common aggregate tests, e.g., fine and coarse aggregate angularity, flat and elongated particles, etc. Ideally, it would have been desirable to select aggregates based on specific (i.e., predetermined) test properties for comparison, but this is not practically possible. To get around this, it was decided to find two sources of aggregates (one manufactured [crushed], and one natural [uncrushed]) and test the aggregates to make sure they were clearly different.

Automated Sand and Gravel of Fort Dodge, Iowa, provided both the coarse and fine natural aggregates used in this study. Martin-Marietta Aggregates of Ames, Iowa, supplied the manufactured (crushed) aggregates used in the study.

Aggregate Testing

The next question is What aggregate properties or parameters need to be characterized and/or measured in the study? Superpave requires two categories of aggregate tests:

1. Consensus properties, which measure critical aggregate characteristics necessary to achieve good performance. These tests are (a) coarse aggregate angularity, (b) fine aggregate angularity, (c) flat, elongated particles, and (d) clay content.
2. Source properties, which are also important to mixture performance but are source specific and relate to the inherent quality of the parent material. These tests include (a) toughness, (b) soundness, and (c) deleterious materials.

Since both the manufactured and natural aggregates are regularly used in HMA production, the source tests were not performed. The consensus tests were performed on both aggregates.

Aggregate Properties

Coarse Aggregates

Unfortunately, the Superpave tests that measure coarse aggregate shape and surface texture are generally rather indirect:

- flat or elongated particles in coarse aggregate (ASTM D4791) and
- determining the percentage of fractured particles in coarse aggregate (ASTM D5821).

Flat and elongated particles impede compaction and consequently affect strength. This test uses a proportional caliper device to determine whether each particle exceeds a specified ratio of maximum to minimum dimension ratio. Most states (81 percent) specify a ratio of 5:1 (*40*). The fractured particles test is performed on aggregates retained on the No. 4 sieve. A fractured particle is defined as a particle with one or more crushed faces, with ASTM specifying a crushed section as having a minimum crushed area of 25 percent of the maximum cross-sectional area of the particle. For the aggregates used in the study, the results of these two tests are shown in Table 8.

TABLE 8 Shape and Surface Texture Properties for Coarse Aggregates

		Fractured Faces (percent mass)		Flat and Elongated (percent mass)	
		1 or more	**2 or more**	**3:1**	**5:1**
Natural	12.5 mm	0	0	0.5	0
	9.5 mm	0	0	0.5	0
	4.75 mm	0	0	2.3	0
Manufactured	12.5 mm	100	100	1.5	0
	9.5 mm	100	100	1.2	0
	4.75 mm	100	100	1.3	0

For the natural aggregates, most of the material had obviously been fractured at one time but had been subsequently worn smooth. There were no freshly fractured faces. There were a few flat particles, but none that were elongated. For the manufactured aggregates, the material was entirely fractured on two or more faces, with a small percentage of flat particles, but none that were elongated.

Fine Aggregates

Conventionally, most state highway agencies control the fine aggregate particle shape and surface texture by specifying a maximum percentage of natural sand in the aggregate blend (*35*). Superpave uses a single test to measure the particle shape and surface texture of fine aggregate: uncompacted void content of fine aggregate (ASTM C1252).

In this test, fine aggregate of a specified gradation is funneled into a cylinder. The amount that is retained in the cylinder is weighed, and the voids are computed using the bulk specific gravity of the fine aggregate. There are three variations to the test: method A (a specified blend), method B (individual sieve size), and method C (gradation as received).

The three methods are not interchangeable, i.e., the results using method A should not be compared with results using methods B or C. In short, method A is specified by Superpave to be the preferred method, and method C is not recommended as fluctuations in gradation during production can significantly influence the value obtained. Only test methods A and B were performed on the materials, and Table 9 below shows the results obtained.

TABLE 9 Shape and Surface Texture Properties of Fine Aggregate

		Manufactured	**Natural**
Method A	Specified Blend	46.7	40.7
Method B	Nos. 8–16	51.2	42.3
	Nos. 16–30	53.2	47.5
	Nos. 30–50	53.1	46.6
	Nos. 50–100	54.8	49

The results indicate that the manufactured and natural aggregates were significantly different in uncompacted void content.

Clay Content

Superpave uses one test to measure the percentage of clay in the aggregate fraction that is finer than the No. 4 sieve: plastic fines in graded aggregates and soils by use of sand equivalent test (ASTM D2419).

For both aggregate sources, the gradation with the highest content of material passing the No. 4 sieve was used. The results, shown in Table 10, were convincing enough to suggest that both were very clean and good aggregates for asphalt mixtures (this is reassuring since they are both being used for asphalt mixes!).

TABLE 10 Clay Content

	Manufactured			Natural		
Nominal maximum size (mm)	**19.5**	**12.5**	**9.5**	**19.5**	**12.5**	**9.5**
Sand equivalent	—	—	95	—	—	91

Specific Gravity

The tests used to determine specific gravity are

- specific gravity and absorption of fine aggregate (ASTM C128) and
- specific gravity and absorption of coarse aggregate (ASTM C127).

Because the study focuses on mix volumetrics, for obvious reasons, these tests were of great importance. These tests were run on duplicate or triplicate specimens (often more!) to try to obtain the specific gravity for each sieve size. The specific gravity test results are listed in Table 11.

TABLE 11 Specific Gravity Test Results Obtained Using ASTM C127 and C128

	Manufactured	Natural
12.5 mm	2.558	2.481
9.5 mm	2.578	2.515
No. 4	2.553	2.519
No. 8	2.591	2.543
No. 16	2.593	2.546

For the fine aggregate passing the No. 8 sieve, tests were performed using a Le Chatelier’s flask following ASTM C188 and C189, except water was used instead of kerosene. This method was used because of the difficulties inherent in getting the relatively single-sized sieved material finer than No. 16 to an identifiable saturated-surface dry condition. The specific gravity test results (averages with standard deviations) are listed in Table 12.

TABLE 12 Specific Gravity Results Obtained Using ASTM C188 and C189

	Manufactured		Natural	
	Average	Standard Deviation	Average	Standard Deviation
No. 30	2.712	0.015	2.648	0.006
No. 50	2.706	0.002	2.638	0.009
No. 100	2.741	0.013	2.664	0.011
No. 200	2.777	0.048	2.673	0.014
P200	2.817	0.035	2.640	0.025

As might be observed, there is quite a difference in values between the two methods, which is due to the fact that the values obtained using Le Chatelier's flask are more an apparent specific gravity than a bulk specific gravity. Trying to get the bulk specific gravity for the individual fractions finer than No. 16 is very difficult using ASTM C128. Hence, while not strictly correct, the averaged results for the sieved source materials were used to calculate the bulk specific gravity for each of the blends, listed in Table 13.

TABLE 13 Calculated Specific Gravity for Each Aggregate Blend

	Gradation	9.5 mm	12.5 mm	19.0 mm
Manufactured	Fine	2.647	2.631	2.624
	Dense	2.628	2.616	2.608
	Coarse	2.612	2.604	2.599
50/50 Blend	Fine	2.613	2.597	2.592
	Dense	2.593	2.585	2.578
	Coarse	2.577	2.573	2.571
Natural Coarse–Manufactured Fine (NCMF)	Fine	2.632	2.607	2.599
	Dense	2.601	2.590	2.580
	Coarse	2.577	2.573	2.570
Natural	Fine	2.580	2.565	2.560
	Dense	2.559	2.554	2.549
	Coarse	2.542	2.544	2.543

The results for the blends show a trend in regard to nominal maximum aggregate size, generally decreasing specific gravity as nominal maximum aggregate size increases. Likewise, as the mixes get finer, the specific gravity increases.

Absorption is not reported because it was not obtained on the sieve sizes where Le Chatelier's flask was used. Absorption is indicative of how much binder the aggregate will absorb; e.g., higher water absorption generally indicates higher asphalt absorption.

Gradations

Three nominal maximum aggregate sizes, 19.0, 12.5, and 9.5 millimeters (0.75, 0.5, and 0.375 inches, respectively) were selected to represent the asphalt mixes commonly used in Iowa. Aggregate fractions were carefully proportioned in the laboratory to meet the selected target gradations shown in Figures 7–

9 and Table 14. One fine, one dense (following the Fuller curve), and one coarse gradation were selected for each nominal maximum aggregate size. Each blend was designed to have the same amount of passing 75 micron (P200) and this was checked using a washed-sieve analysis. The material on each sieve was weighed to the nearest 0.1 gram.

Aggregate Blends

Table 1 (page 6) shows the intended laboratory test plan along with the completed test schedule. As shown, the original test matrix specifies nine blends times nine gradations times five asphalt contents times two (replicate) specimens = 810 specimens. After completing testing of the first 90 specimens, time and material limitations and the clarity of the results to date dictated that the scope of testing be reduced to four blends (a total of 360 specimens). The four blends selected are

1. manufactured—each gradation is 100 percent crushed material (coarse and fine);
2. 50/50 blend—each gradation is a blended 50 percent crushed, 50 percent natural on each sieve size;
3. manufactured fine-natural coarse—the material passing the No. 4 sieve was 100 percent crushed and the material retained 100 percent natural. The coarse (natural) aggregate was washed to ensure that the P200 material was obtained entirely from the crushed aggregates; and
4. natural—each gradation is 100 percent natural material (coarse and fine).

It was believed that these four blends would provide enough information to evaluate the effects of gradation and shape for both the fine and coarse aggregates.

Summary

The first step in the project was to select sources of manufactured and natural coarse and fine aggregates to be used in the study. Once this was done, the next step was to characterize the aggregates through testing to measure differences. The Superpave aggregate consensus tests were performed on both the manufactured and natural aggregates. The results obtained are reported in Tables 8, 9, and 10. The bulk specific gravity was calculated for each of the 36 blends, and these results are presented in Table 13.

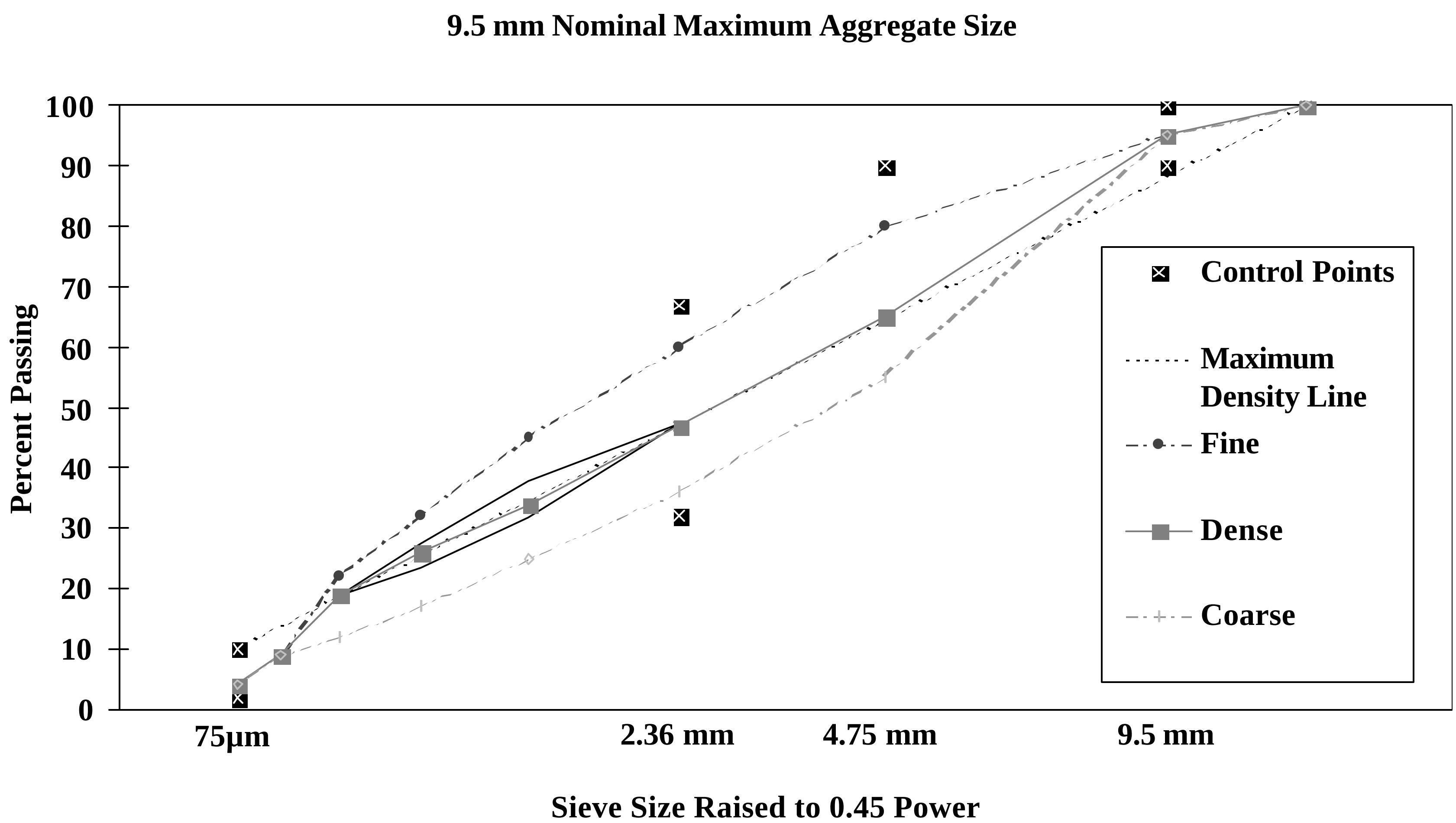

FIGURE 7 9.5-millimeter Nominal Maximum Size Gradations Used in Study

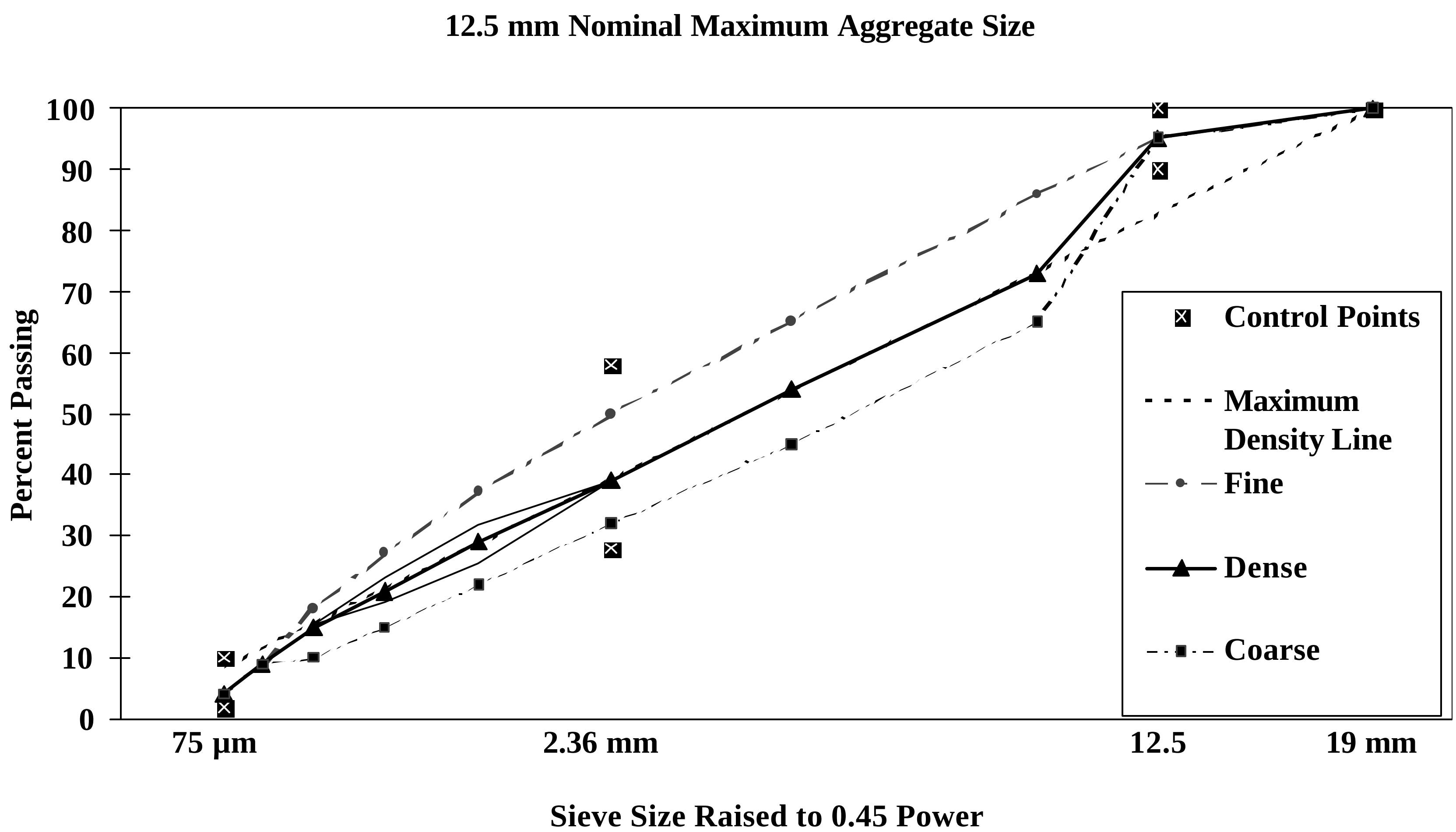

FIGURE 8 12.5-millimeter Nominal Maximum Size Gradations Used in Study

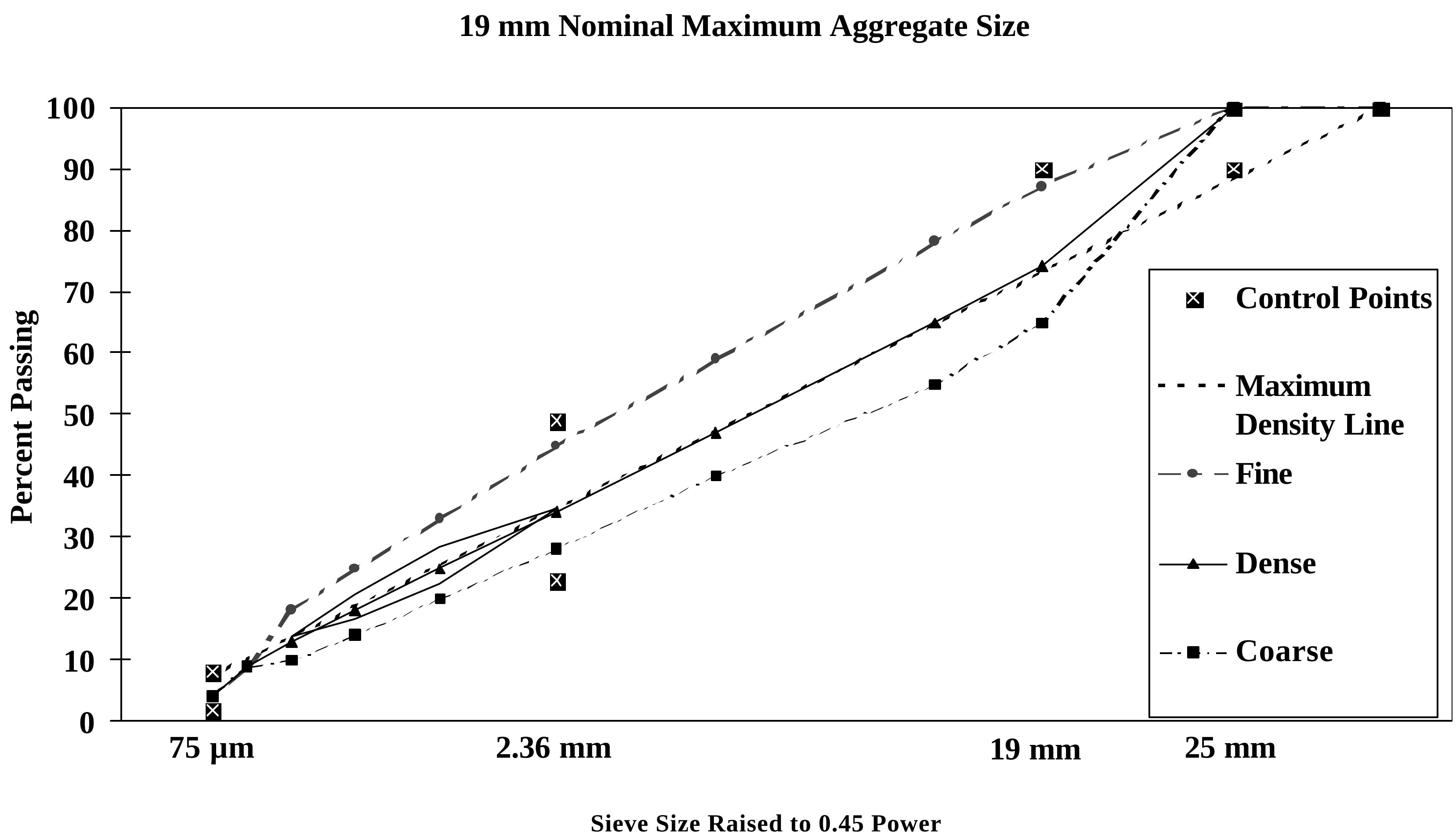

FIGURE 9 19.0-Millimeter Nominal Maximum Size Gradations Used in Study

TABLE 14 Aggregate Gradations

Sieve No. (mm)	Percent Passing								
	9.5-mm NMAS			12.5-mm NMAS			19.0-mm NMAS		
	Fine	Dense	Coarse	Fine	Dense	Coarse	Fine	Dense	Coarse
19.0	100	100	100	100	100	100	100	100	100
12.5	100	100	100	95	95	95	87	74	65
9.5	95	95	95	86	73	65	78	65	55
4.75	80	65	55	65	54	45	59	47	40
2.36	60	47	36	50	39	32	45	34	28
1.18	45	34	25	37	29	22	33	25	20
0.600	32	26	17	27	21	15	25	18	14
0.300	22	19	12	18	15	10	18	13	10
0.150	9	9	9	9	9	9	9	9	9
0.075	4	4	4	4	4	4	4	4	4

4 METHODOLOGY

Once the materials had been selected and characterized, the focus shifted toward the methodology to be used in laboratory testing. From the outset of the project, the approach was to follow Superpave mix design procedures (*41*), to follow the applicable British Standards for the use of the NAT, and to streamline test wherever possible. The same specimens would be used (and reused where possible) throughout the testing, for bulk specific gravity testing, NAT testing, and for theoretical maximum specific gravity determination. Thus, in presenting the testing methodology, any deviations from either the Superpave mix design protocol or applicable British Standards are highlighted and discussed to ensure that there is no confusion.

Preliminary Issues

Prior to beginning testing, the two greatest concerns were the unfamiliarity with the NAT repeated load triaxial (RLT) test apparatus and compaction of the specimens in the Superpave Gyratory Compactor (SGC). From the literature search it was determined that the Transport Research Laboratory (TRL) found the NAT equipment promising as a laboratory test for assessing deformation resistance, and that it was fairly easy to use (*17*). However, this was only the second NAT imported into the United States, and the first configured with RLT test capability. Hence, some basic questions with the test apparatus had to be answered prior to laboratory testing:

1. What test conditions (temperature, stress regime, duration) would be used?
2. What information would the RLT provide?
3. Was it compatible with asphalt specimens compacted in the Superpave Gyratory Compactor?
4. Would sawing and polishing be required prior to testing the SGC specimens?
5. How much time and effort would be involved per cell of testing?

Specimen compaction also posed some difficult questions:

1. Would compaction be fixed (at a given number of gyrations in the SGC, N_{gyr}) or would a fixed air void content be targeted?
2. What level of compaction would be used?

Efficiency answered the first question—it was just not practical to try to get all specimens to a fixed air voids (four percent). It would have required considerable time and would have used up a large quantity of material. For a study examining aggregate-related factors, using different compaction levels would possibly confuse the issue, as the crushed material would undoubtedly require more compactive effort to achieve the same void content. This additional compactive effort, combined with the crushed aggregate surface texture, would skew the results. Increasing the compaction could also lead to particle crushing, which would further bias the results.

Once it was decided to use a single compaction level, the next step was to select which level to use. For the study, specimens would be compacted to 109 gyrations plus a five-gyration leveling load. This is the required N_{des} compaction for traffic between 10 and 30 million equivalent single axle loads (ESALs), a common design traffic level used for interstate highways in Iowa.

Pilot Study

To get familiar with the new equipment, a pilot study was undertaken to ascertain the capabilities and limitations of the NAT. As presented in the literature review, there are currently no standard test conditions for the RLT test. Temperatures of 40, 45, and 50 degrees C, (104, 113, and 122 degrees F, respectively) were used in conjunction with confining pressures of 35, 70, and 100 kPa (5, 10, and 14.3 psi, respectively). Initially, a deviator stress of 250 kPa (35.7 psi) was used; this was raised to 300 kPa (42.9 psi), the limit of the equipment, when a new source of air pressure was installed. Of critical importance was determining the number of cycles to be used in each test. The load frequency is fixed at 2 hertz, hence there would be 1800 load applications in one hour. The maximum test duration of the equipment is 10,000 cycles. Because of the number of specimens to be tested, it was deemed imperative that the test be no longer than one hour. It was also necessary to determine the conditioning time for a specimen to get to test temperature. From the pilot study the following information was learned:

1. Test conditions of 45 degrees C (113 degrees F), 17 kPa (2.4 psi) confining stress, 300 kPa (42.9 psi) deviator stress, and a test duration of 1800 cycles (one hour) would be used. It takes approximately 125 minutes for the specimens to get to test temperature; therefore, 130 minutes was used as the conditioning time for the study.
2. The RLT measures vertical strain and computes stiffness. There is no measure of volumetric strain.
3. The RLT is compatible with SGC specimens. However, specimens of normal height (115 millimeters) are the upper limit of the equipment *as configured* and are awkward to test. This limits the maximum practical height to diameter ratio to about 0.75, which is below the conventionally accepted minimum ratio of 1:1 for triaxial testing.
4. The specimens would not be cut and polished; however, they would be lubricated with silicon grease prior to testing.
5. Based on the conditioning time of 130 minutes and assuming an average time of 10 minutes to remove and replace test specimens, five to six specimens could be tested in a typical day.

The conditions used for testing are summarized in Table 15.

TABLE 15 Test Conditions Used in the Study

Test Property	Test Conditions
Temperature	45 degrees C (113 degrees F)
Deviator stress	300 kPa (42.9 psi)
Confining stress	17 kPa (2.4 psi)
Number of repetitions	1800 cycles (1 hour)
Specimen ends	Unsawn, lubricated
Preconditioning	2 hours at test temperature

The awkwardness of testing 115-millimeter (4.53-inch), 4700-gram (10.4-pound) specimens made it desirable to use a different size of specimen. Previous research (*42*) and consultation with the Iowa DOT bituminous engineer and his staff indicated that the density of the SGC compacted HMA would not be significantly affected if the specimen height was decreased to 75 millimeters (2.95 inches) and weight to 3375 grams (7.44 pounds).

Laboratory Testing Protocol

The protocol used for laboratory testing followed AASHTO standards wherever possible. However, because there were some deviations from convention, for discussion purposes, the laboratory work is broken down into distinct steps:

1. batching;
2. mixing, aging, and compaction;
3. pre-NAT bulk specific gravity;
4. NAT testing;
5. post-NAT bulk specific gravity; and
6. theoretical maximum specific gravity.

The laboratory process is shown graphically in a flowchart (Figure 10).

Batching

Prior to testing, the aggregates had been dried, sieved, and stored in 20-gallon containers. Once a gradation blend was selected, the first step was to determine the quantity of filler (material passing the 75-micron sieve) contained in that blend. To do this, a washed-sieve analysis was performed following the procedures of AASHTO test method T11-91, on two 1000-gram (2.2-pound) samples. The test results were averaged, and if the difference was more than 0.5 percent, a third test was performed.

Once the percent of filler was determined, ten specimens (two at each asphalt content) were blended as shown in Table 16. Specimens were heated in an oven overnight to approximately 160 degrees C (320 degrees F). The asphalt was heated at 147 degrees C (297 degrees F) until it was sufficiently fluidal for mixing.

TABLE 16 Batch Aggregate Weights Used in Laboratory Testing

Asphalt Content (by weight of mix)	Weight of Blended Aggregate
4	3240 grams (7.14 pounds)
5	3206.3 grams (7.07 pounds)
6	3172.5 grams (6.99 pounds)
7	3138.8 grams (6.92 pounds)
8	3105 grams (6.85 pounds)

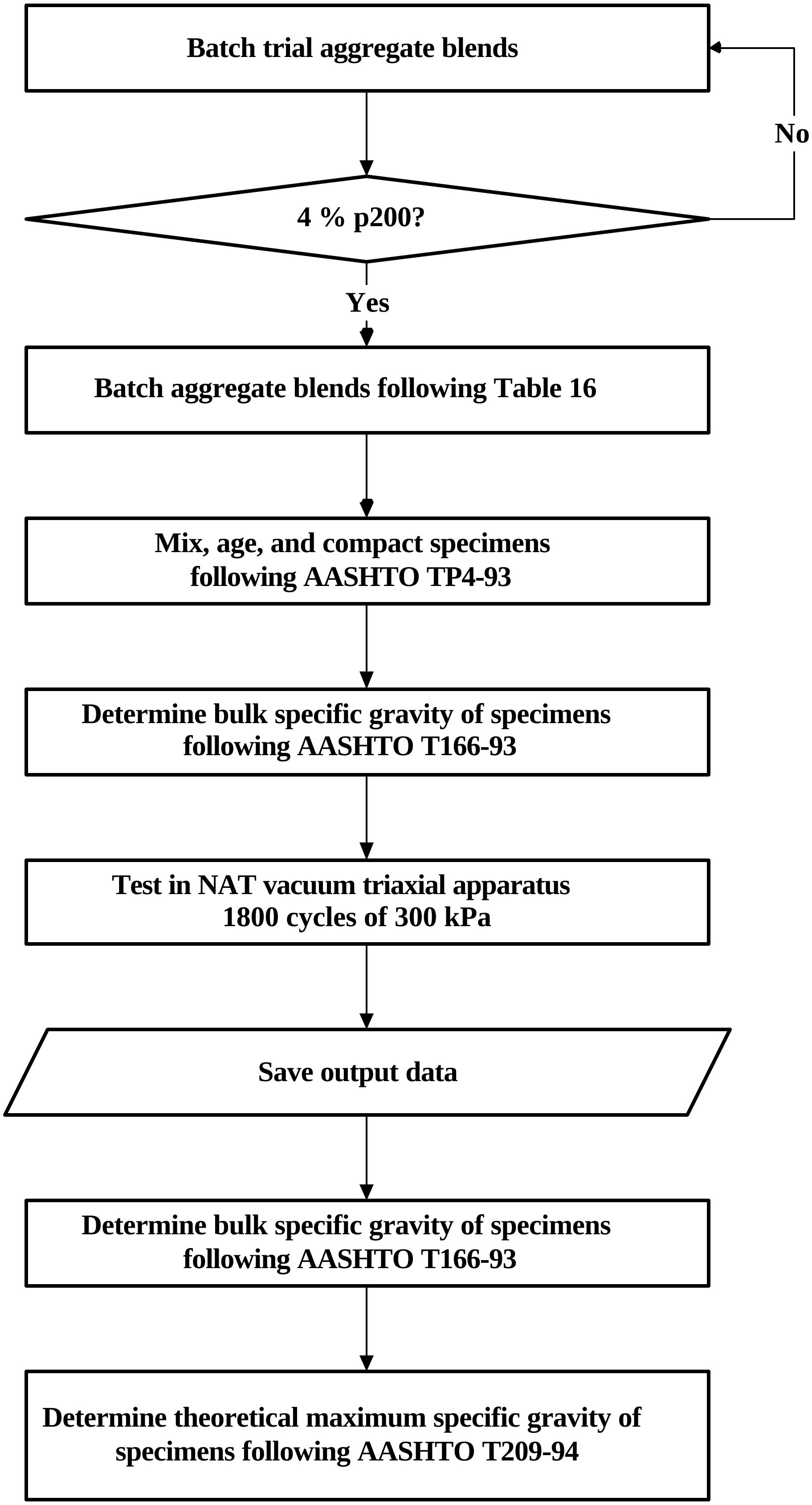

FIGURE 10 Flow Chart of Laboratory Testing

Mixing, Aging, and Compaction

Mixing, aging, and compaction were performed in accordance with AASHTO TP4-93. The viscosity of the binder targeted a mixing temperature of 147 degrees C (297 degrees F) and compaction temperature of 135 degrees C (275 degrees F). The aggregates were placed into a heated mixing bowl and dry mixed by hand. The asphalt was added, then the asphalt-aggregate mixture was mixed mechanically for 30–45 seconds (until a uniform coating was observed). The mix was then transferred to a pan and aged for two hours in an oven at 135 degrees C (275 degrees F). After an hour the mix was stirred to ensure uniform heating and aging.

The specimens were compacted to 109 gyrations in the SGC then allowed to cool overnight. Some of the "rich" mixes required using two sets of papers in the mold to prevent the compacted specimen from sticking to the ram. Once cooled, the bulk specific gravity of the compacted specimens was obtained following AASHTO T166-93. The specimens were then "air dried" back to within 1 gram of their original weight.

NAT Testing

Prior to testing, the specimens were conditioned in NAT for 130 minutes to ensure that they were equilibrated at the test temperature of 45 degrees C (113 degrees F). NAT requires specimen heights to the nearest millimeter; the SGC provides height data to a tenth of a millimeter. After checking several specimens with a micrometer, it was decided to use the SGC height data and round to the nearest millimeter.

Once the specimens were at test temperature, the platens of the apparatus were coated with a thick layer of silicon-Teflon grease. The specimen was placed on the bottom platen, the rubber membrane slid over the specimen, and secured with an O-ring. The top platen was set in place and secured with an O-ring. Then the jacketed specimen was placed in the temperature chamber, and the vacuum hose was connected. The vacuum of 17 kPa (2.4 psi) drew the membrane tight, and any wrinkles were smoothed out. Then the apparatus was centered in the load frame, the cross head adjusted to the correct height, and linear variable differential transformers (LVDTs) centered for testing. With practice, the procedure can be done very quickly, only taking a few minutes. There is a two-minute period of load preconditioning prior to the test beginning. After this, the specimen receives 1800 applications of a 300 kPa (42.9 psi) load, and the accumulated axial strain is measured.

Once the test is complete, the specimen is carefully removed and is allowed to cool to room temperature, the platens and membrane are cleaned and wiped dry, and the next test is started.

Post-NAT Testing

After cooling, the bulk specific gravities of the specimens were again measured in accordance with AASHTO T166-93. There usually was not a significant difference between the pre-NAT and post-NAT bulk specific gravity. The specimens were then placed in a pan and heated for approximately two hours at 135 degrees C (275 degrees F) to soften them up to break prior to determining their theoretical maximum specific gravity following AASHTO T209-94.

Summary

Developing a consistent, rigorous, and, most important, usable test protocol was a fundamental task in the study. It was important to follow existing specifications wherever possible yet at the same time perform the testing on schedule.

AASHTO specifications were followed with one notable exception in that the mass of the SGC specimens was 3375 grams (7.44 pounds), instead of 4500–4700 grams (9.92–10.23 pounds). The applicable British Standards calling for specimen ends to be sawn and polished were not followed as that would have been time consuming and would have created difficulties with determining the theoretical maximum specific gravity of the test specimen. Compacted specimens ranged in height from 75 to 87 millimeters (2.95 to 3.43 inches).

5 ANALYSIS OF TEST DATA

In this section, the results obtained from the laboratory testing are analyzed and discussed. The final conclusions are developed and presented.

Definitions

The VMA is the volume of intergranular void space between the coated aggregate particles of a compacted paving mixture, which includes the air voids and volume of the asphalt not absorbed into the aggregates.

Two additional definitions are crucial to the ensuing discussions and must be clearly differentiated: minimum VMA and critical VMA. Specifications and literature abound with requirements for "minimum VMA." Previous studies into VMA dating back to McLeod's original paper (*2*) make reference to "minimum VMA" and draw conclusions and make recommendations based on consideration of this parameter. The authors believe that there is some confusion within the industry in this regard and seek to clearly differentiate between the different flavors of VMA.

Minimum VMA

For the purposes of this study and report, the term "minimum VMA" is defined to indicate the smallest VMA measured on a given aggregate blend, when compacted with a given energy over a range of binder contents. This is a statement of the volumetric state of a mixture under certain conditions—*it makes no statement as to the competence or suitability of the mixture at that state.*

In Figure 11, it can be seen that a representative aggregate blend used in this study (the natural coarse-natural fine 12.5-millimeter coarse gradation), compacted to 109 gyrations in SGC over a range of binder contents (4 to 8 percent), demonstrates a minimum VMA of approximately 12 percent at 5.4 percent asphalt content.

Specified VMA

As distinct from minimum VMA, as defined above, Superpave (*40*), the Asphalt Institute (*36*) and others specify that the VMA of a design mixture shall not be less than a specified "minimum VMA." This minimum VMA refers to a suggested relationship between VMA and nominal maximum aggregate size originally proposed by McLeod in 1959 (*2*). McLeod suggested that a mixture with a VMA less than that specified would have insufficient "space" or "free volume" to contain the volume of binder coating the aggregate particles and the volume of air voids deem appropriate for satisfactory performance. This relationship has been modified over the years but remains essentially the same as originally proposed. Table 17 shows the current Superpave specification of this "minimum VMA" (*40*).

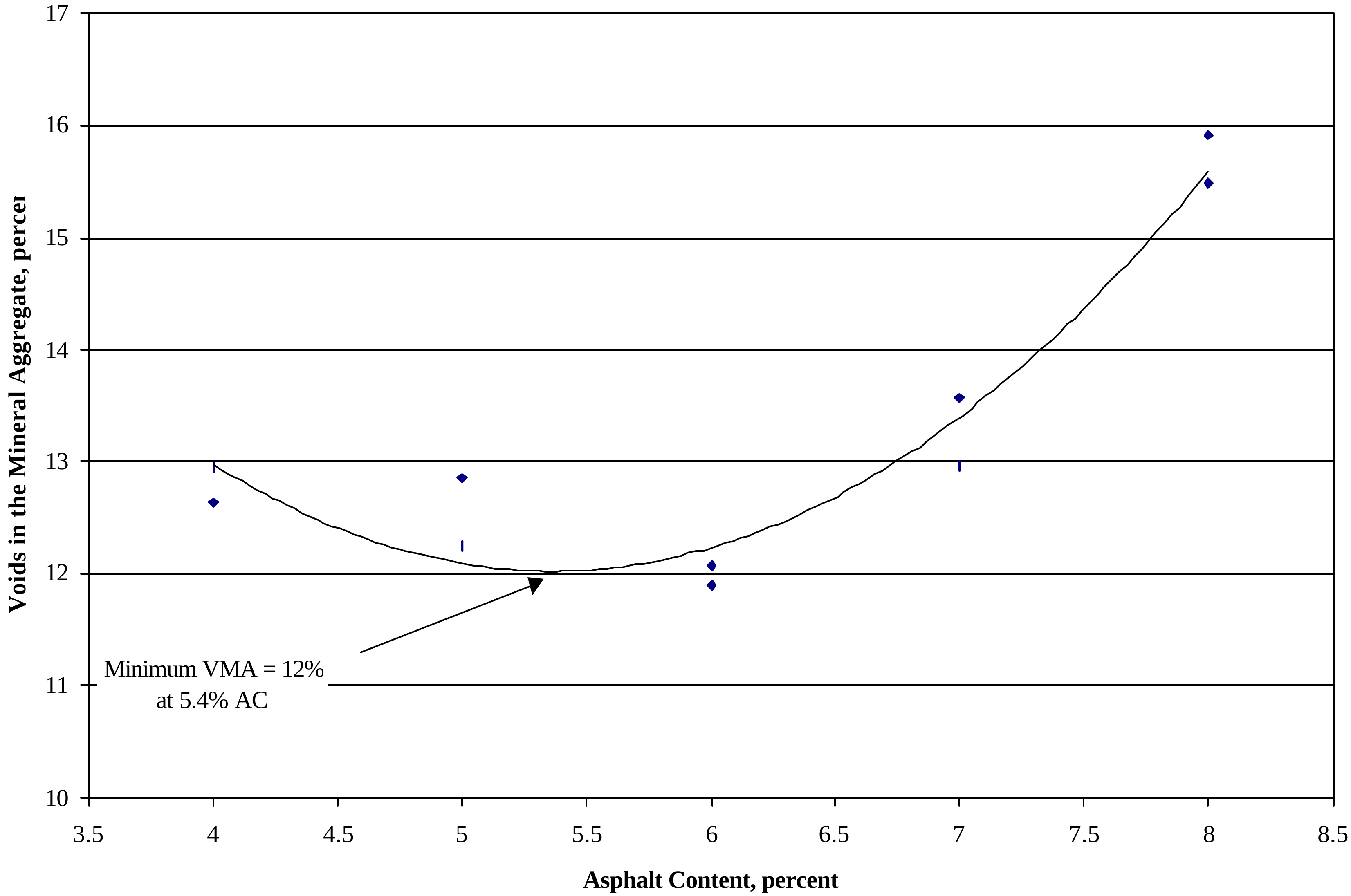

FIGURE 11 Defining "Minimum VMA"

TABLE 17 Superpave Specified VMA

Nominal Maximum Size		Specified (minimum) VMA (percent)
Metric	**U.S. Customary**	
9.5 mm	3/8 inches	15.0
12.5 mm	0.5 inches	14.0
19.0 mm	0.75 inches	13.0
25.0 mm	1 inch	12.0
37.5 mm	1.5 inches	11.0
50.0 mm	2 inches	10.5

Critical VMA

By implication, a mixture with a VMA larger than the specified VMA should be sound, while one with a VMA less than that specified is expected to be unsound. This project seeks to measure the VMA at which mixtures in fact transition from sound to unsound. This identified VMA is referred to as the "critical VMA."

The first objective of this project is to determine whether the "specified VMA" correctly identifies the "critical VMA" of HMA mixtures. Indeed, this is our evidence against which the McLeod hypothesis is to be tested.

In most cases, the specified (minimum) VMA is intended to address the problem of strength and stability of the mixture. However, there is some discussion within the industry about the possibility of defining a "maximum VMA" designed to address the question of durability. This concept is not addressed in this project.

Laboratory Results

The analysis of test data included a preliminary step to determine the critical transition (i.e., the condition at which the mixture is identified by testing to transition from sound to unsound behavior). The volumetric properties of mixtures at the point of transition are thereafter identified and subjected to statistical analysis.

Preliminary Analysis of Results

Once the laboratory testing was complete, the test data were analyzed to determine the critical volumetric properties for each of the 36 aggregate blends. The first step of the analysis was to determine the critical transition asphalt content of the compacted HMA mixture based on a visual analysis of the NAT results. To show how this was done, the test results for the three 19-millimeter NMAS crushed aggregate blends are shown in Figure 12. The critical transition point was the asphalt content at which the mix became unsound, i.e., where the axial strain rate began to increase dramatically. Examining Figure 12, the critical asphalt contents of the three mixes are 6.6 for the coarse, 6.3 for the dense, and 6.9 for the fine-graded mix. Five of the 36 mixes did not become unsound over the range of asphalt contents used in the study. For each of the thirty-one mixes that became plastic (i.e., unsound), the volumetric properties were calculated at the critical point. Whereas McLeod specified VMA at five percent air voids and Superpave at four percent air voids, the critical VMA identified in this project is defined at whatever air content was measured at the point that the mixture became unstable.

This procedure was performed for each of the 36 blends, and the critical-state volumetric properties are presented in Table 18. As shown in Table 18, five of the 36 gradations did not become unsound over the range of asphalt contents used in the study (four to eight percent).

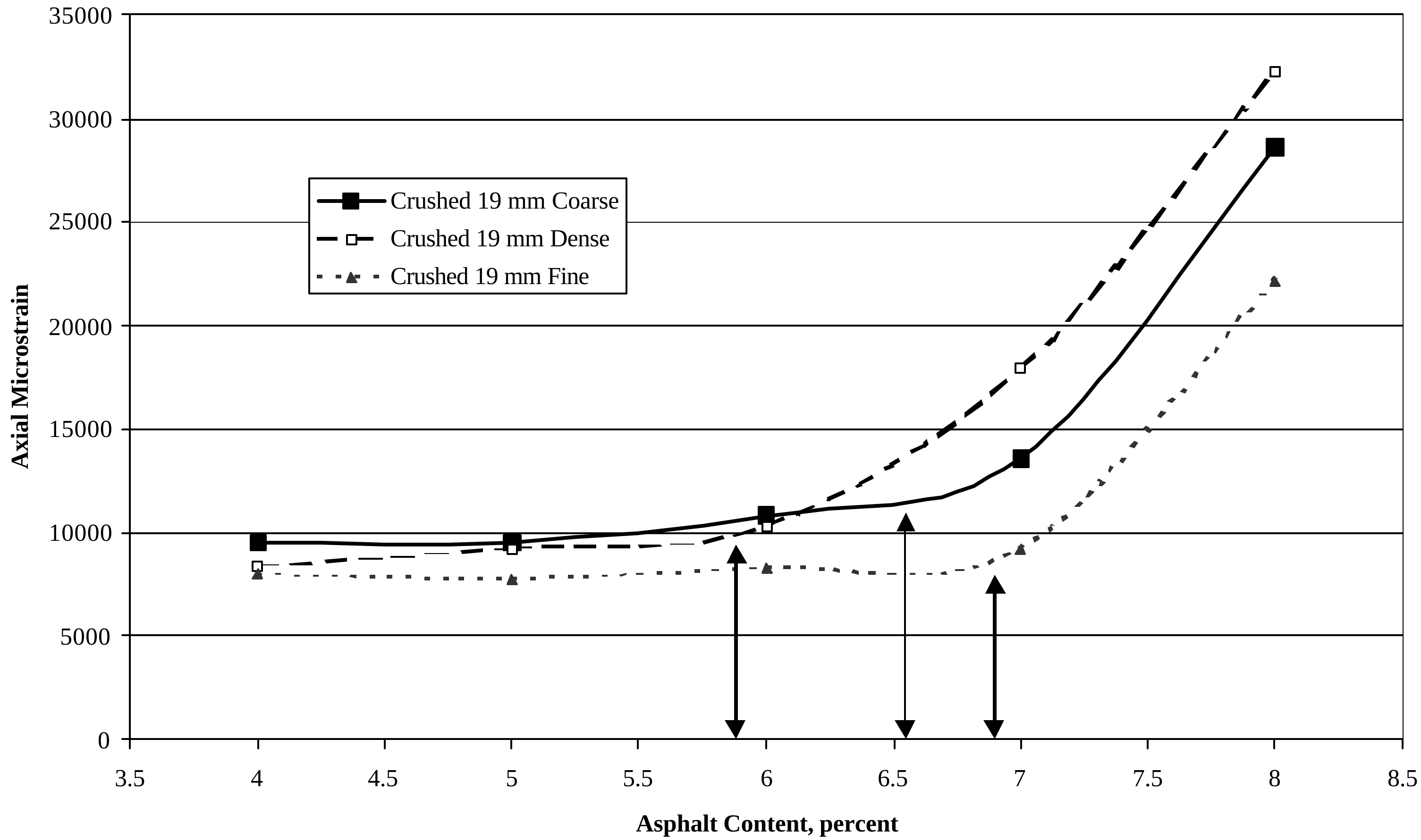

FIGURE 12 NAT Results Used for Determining Critical Transition

TABLE 18 Summary of Measured Critical State Volumetric Parameters

Gradation	CA/FA	NMAS (mm)	Pb_{crit}	V_a	VMA	VFA	FT
Coarse	50/50	9.5	6.2	3.0	13.6	77.7	9.1
Coarse	50/50	12.5	5.4	3.0	11.6	74.5	7.7
Coarse	50/50	19	4.8	2.9	10.0	70.5	6.4
Dense	50/50	9.5	6.4	3.2	13.7	76.3	8.6
Dense	50/50	12.5	5.6	1.6	11.7	86.4	7.7
Dense	50/50	19	4.4	4.8	11.1	56.5	5.2
Fine	50/50	9.5	N/R				
Fine	50/50	12.5	5.9	3.4	14.3	76.1	7.8
Fine	50/50	19	5.1	2.2	10.9	79.8	6.3
Coarse	M/M	9.5	6.2	2.8	13.8	79.9	9.2
Coarse	M/M	12.5	6	1.6	12.1	86.9	9.3
Coarse	M/M	19	5.4	3.0	11.9	74.3	8.0
Dense	M/M	9.5	7.1	1.1	16.1	93.0	9.5
Dense	M/M	12.5	6	3.2	13.7	76.3	8.2
Dense	M/M	19	5.7	2.2	12.1	82.0	8.1
Fine	M/M	9.5	N/R				
Fine	M/M	12.5	N/R				
Fine	M/M	19	6.4	3.4	15.0	77.2	8.5
Coarse	NCMF	9.5	6.3	3.5	13.7	74.4	8.5
Coarse	NCMF	12.5	5.6	2.5	11.4	78.4	7.9
Coarse	NCMF	19	5.1	2.3	9.6	76.4	6.6
Dense	NCMF	9.5	7.2	1.2	14.6	91.8	9.4
Dense	NCMF	12.5	5.8	2.2	12.2	82.3	7.6
Dense	NCMF	19	5.3	3.1	11.2	72.5	6.6
Fine	NCMF	9.5	N/R				
Fine	NCMF	12.5	N/R				
Fine	NCMF	19	6.0	2.9	13.2	78.1	7.4
Coarse	N/N	9.5	5.3	2.4	11.4	78.6	7.6
Coarse	N/N	12.5	5.5	2.7	12.3	78.2	8.8
Coarse	N/N	19	4.8	2.4	8.8	73.2	6.0
Dense	N/N	9.5	5.4	2.6	12.6	79.7	6.9
Dense	N/N	12.5	5	2.1	10.6	80.3	6.7
Dense	N/N	19	4.5	3.2	9.3	65.9	5.1
Fine	N/N	9.5	5.1	5.6	14.6	61.4	5.7
Fine	N/N	12.5	5.3	3.2	12.9	75.5	6.9
Fine	N/N	19	5.1	2.8	11.0	74.7	5.9

Statistical Analysis

The first question to be posed and answered is whether the specified VMA values given in Table 17 adequately discriminate between sound and unsound mixtures. Figure 13 shows the relationship between the specified and critical VMA identified in this project. It can be seen that only three out of 28 results exceed the specified values. The implications of these results can be summarized as follows:

- A mixture, A (see Figure 13), compacted to the design degree of compaction exhibits a VMA of 15 percent. This exceeds the specified minimum value of 13 percent for a 19-millimeter gradation. All other factors aside, this would be deemed an acceptable mixture. *However, it should be realized that if this mixture were to be "overcompacted" to 14 percent VMA, it would* ***still*** *be deemed acceptable even though it has here been identified to be unstable at any magnitude of VMA less than 15 percent.*
- A different mixture, B, compacted to the design degree of compaction exhibits a VMA of 10 percent. This does not meet the specified minimum VMA requirements and would be rejected as unacceptable. *However, this mixture would, in fact , exhibit stable behavior.*

As previously noted, one important difference between the two sets of data shown is that the values from Table 17 are based upon an air void content of 4 percent, while the values obtained from this project have air void contents in the range 1.9 to 4.0 percent. It should be noted that an earlier edition of Table 17 published in the Asphalt Institute MS-2 provided critical (minimum) VMA values for a range of air void contents (3, 4, and 5 percent). Because VMA = $V_a + V_{\mathrm{be}}$, (or VMA = air void content + effective binder content) and the Asphalt Institute table referenced implied a constant effective binder content for all values of air void contents, it may be inferred that the effective binder content should be the more critical parameter. Figure 14 shows the effective binder content implied by Table 17 above with those obtained from this project in Table 18. In this case, the reliability of the criterion is 10 out of 28.

Clearly, current design criteria are *not* robust predictors of the threshold between sound and unsound performance. In the following sections, each of the relevant volumetric parameters, V_a, VMA, V_{be}, and VFA will be examined in the light of the aggregate properties and results obtained.

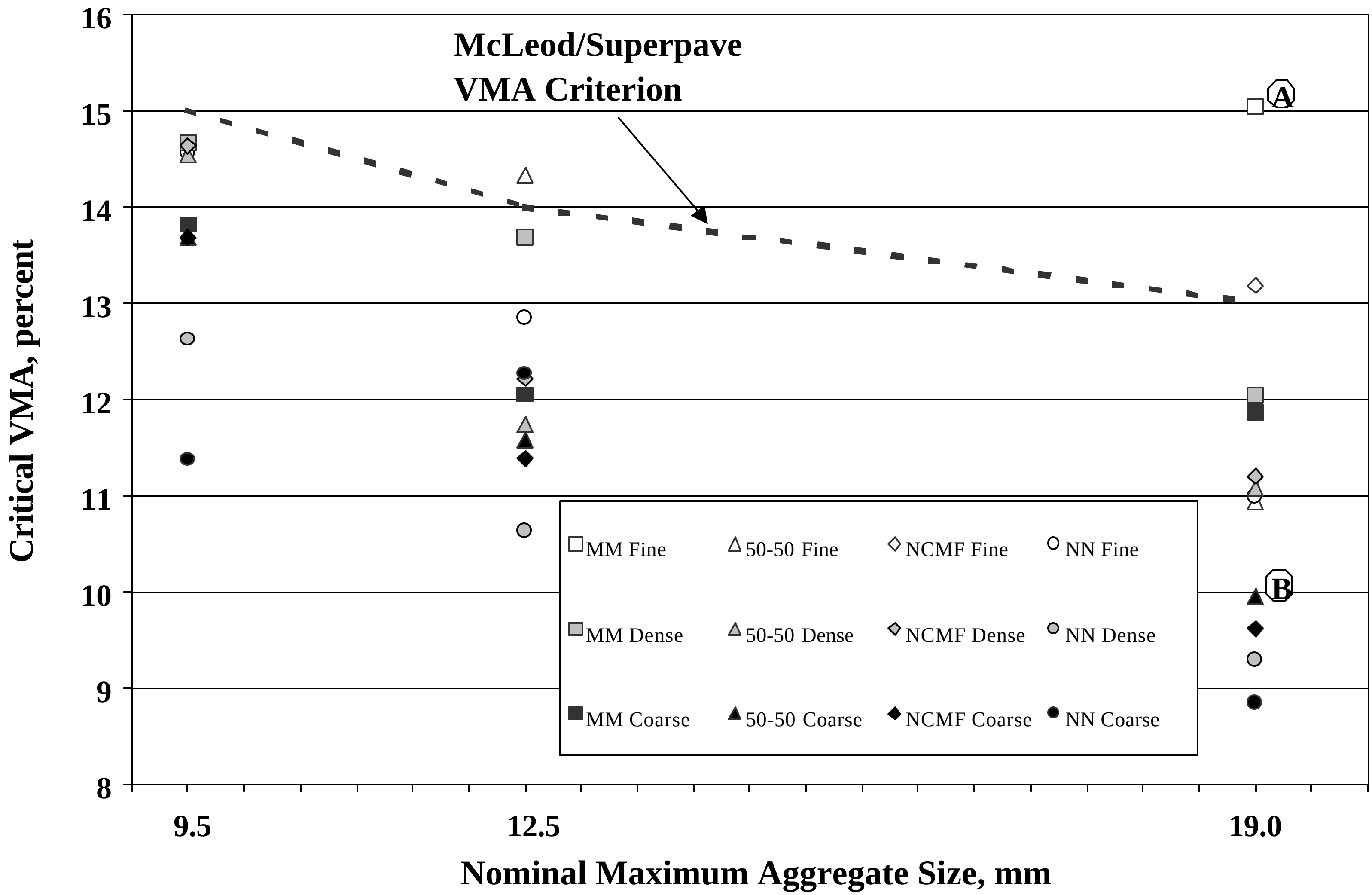

FIGURE 13 Observed Versus McLeod/Superpave Critical VMA

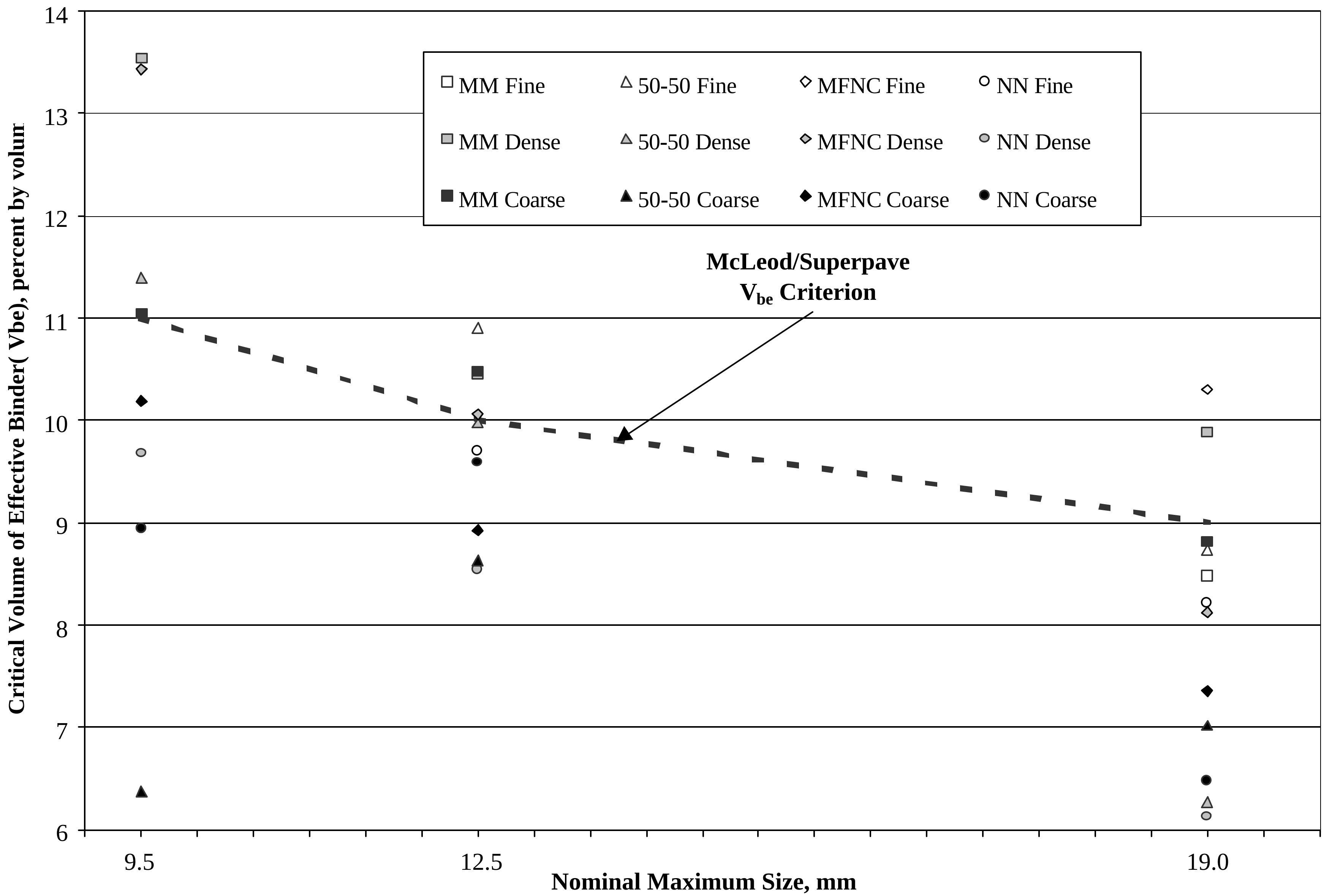

FIGURE 14 Observed Versus McLeod/Superpave Critical V_{be}

Voids in the Mineral Aggregate

It is hypothesized that the VMA at which a mixture becomes unsound (i.e., the critical VMA) is a function of aggregate properties. The current criteria for critical VMA (Table 17) are based solely on the nominal maximum size of the aggregate. Based on anecdotal evidence and personal observation, it has long been felt in the industry that other factors such as aggregate shape and texture must play a part. Furthermore, McLeod stated that his recommendations were based on "dense gradations"; however, dense gradations have become less and less common over the years, and under Superpave they are effectively impossible because of the presence of the so-called "restricted zone." A dense gradation is generally defined as following closely on the Fuller maximum density line.

In Iowa, the DOT has for many years relied on the use of film thickness to limit binder content. While film thickness is primarily a function of the binder content, it is also a function of the surface area of the aggregate blend. Surface area is not a measured quantity but is computed based on surface area coefficients for each size fraction of the aggregate. Consequently, surface area (as defined) is a possible factor in the determination of a critical VMA.

This leads to the hypothesis that the critical VMA in a mixture is a function of various aggregate properties, or

$$VMA_{crit} = \boldsymbol{j}\left(NMAS, CAPC, FAPC, FM, SA\right) + \boldsymbol{e} \quad , \tag{1}$$

where

NMAS = Nominal Maximum Aggregate Size (in millimeters) ,
CAPC = Coarse Aggregate Percent Crushed ,
FAPC = Fine Aggregate Percent Crushed ,
FM = Fineness Modulus (ASTM C33) , and
SA = Surface Area (Asphalt Institute, MS-2).

An ANOVA analysis of the data in Table 18 was performed to identify the significance and quality of the influence of these factors on the critical VMA identified in each mixture tested (see Table 19).

TABLE 19 ANOVA Results for VMA Versus NMAS, CAPC, FAPC, FM, and SA

Source	Sum of Squares	Degrees of Freedom	Mean Squares
Model	76.133	27	2.820
Intercept	4184.617	1	4184.617
FM	51.150	8	6.394
CAPC	12.993	2	6.496
FAPC	9.103	1	9.103
FM x CAPC	2.073	11	0.188
FM x FAPC	0.814	5	0.163

These results indicate that only three of the factors (fineness modulus, CAPC, and FAPC) and two interactions (FM x CAPC and FM x FAPC) are significant at the 5 percent level. The NMAS and surface area are identified as being of no statistical significance.

In order to test these results, it was decided to perform a linear regression analysis of (1) VMA versus log (NMAS), i.e., the original McLeod hypothesis, and (2) VMA versus FM, CAPC, and FAPC. (The interaction factors were dropped at this point since their contributions to the variance were so small, MS < 0.2.)

The first of these regressions, VMA = $\phi(\log_{10}[\text{NMAS}])$,

$$VMA = 22.605 - 9.238\,log_{10}(NMAS) \qquad r^2 = 0.472 \qquad see = 1.220 \quad , \tag{2}$$

yields the results in Table 20.

TABLE 20 Regression Results of McLeod VMA Versus NMAS Relationship

Model	Sum of Squares	Degrees of Freedom	Mean Square	*F*
Regression	37.438	1	37.438	25.155
Residual	38.695	26	1.488	
Total	76.132	27		

This indicates that the observed relationship between measured critical VMA and nominal maximum aggregate size alone is tenuous at best ($r^2 = 0.47$). Comparing the predicted results using equation (2) against the specified values in Table 17, the results in Table 21 are obtained.

TABLE 21 Comparison of Predicted and McLeod/Superpave Critical VMA

Nominal Maximum Aggregate Size		Critical VMA	
Metric	U.S. Customary	Specified (Table 17)	Predicted (Equation 2)
9.5 mm	0.375 inches	15.0	13.6
12.5 mm	0.5 inches	14.0	12.5
19.0 mm	0.75 inches	13.0	10.8
25.0 mm	1 inch	12.0	9.7[a]
37.5 mm	1.5 inches	11.0	8.1[a]
50.0 mm	2 inches	10.5	6.9[a]

[a]These values are extrapolated beyond the range of NMAS tested.

It is clear that there is a significant difference between the two sets of numbers. However, it should be recalled that the specified values are specifically set to allow for an air void content of four percent. The measured values obtained by testing do not contain four percent air voids (Table 19), being deficient by about 1.5 percent in most cases.

The second regression analysis, $VMA_{crit} = ø(FM, CAPC, FAPC)$, yields the results shown in Table 22. From this analysis, we note that the adjusted $r^2 = 0.88$ and the standard error of estimate (s.e.e.) = 0.58. This is a significant improvement on that obtained previously. The resulting predictive relationship is thus

$$VMA_{crit} = 26.20 - 3.34FM + 0.0129CAPC + 0.0155FAPC \quad r^2 = 0.88 \quad see = 0.58 \quad . \tag{3}$$

The meaning of this predictive equation must be clearly stated. It predicts the magnitude of the critical VMA for the mixtures tested and compacted at 109 gyrations of the SGC. Figure 15 shows graphically the very good fit between predicted and observed critical VMA for the data set studied. However, it must be noted that the air void content is not constant. As observed above, the effective binder content, V_{be}, comprises a concomitant significant variable and should be equally investigated.

TABLE 22 Regression Results for $VMA_{crit} = ø(FM, CAPC, FAPC)$

22A Summary Output

	Regression Statistics
Multiple *R*	0.9454
R square	0.8938
Adjusted *R* square	0.8805
Standard error	0.5794
Observations	28

22B ANOVA

	df	**SS**	**MS**	***F***	**Significance *F***
Regression	3	67.7819	22.5940	67.3143	0.0000
Residual	24	8.0556	0.3356		
Total	27	75.8375			

	Coefficients	**Standard Error**	***t*-statistic**	***P*-value**	**Lower 95%**	**Upper 95%**
Intercept	26.1999	1.1447	22.8879	0.0000	23.8373	28.5624
FM	–3.3352	0.2550	–3.0785	0.0000	–3.8616	–2.8089
CAPC	0.0129	0.0030	4.2637	0.0003	0.0067	0.0192
FAPC	0.0155	0.0030	5.2465	0.0000	0.0094	0.0217

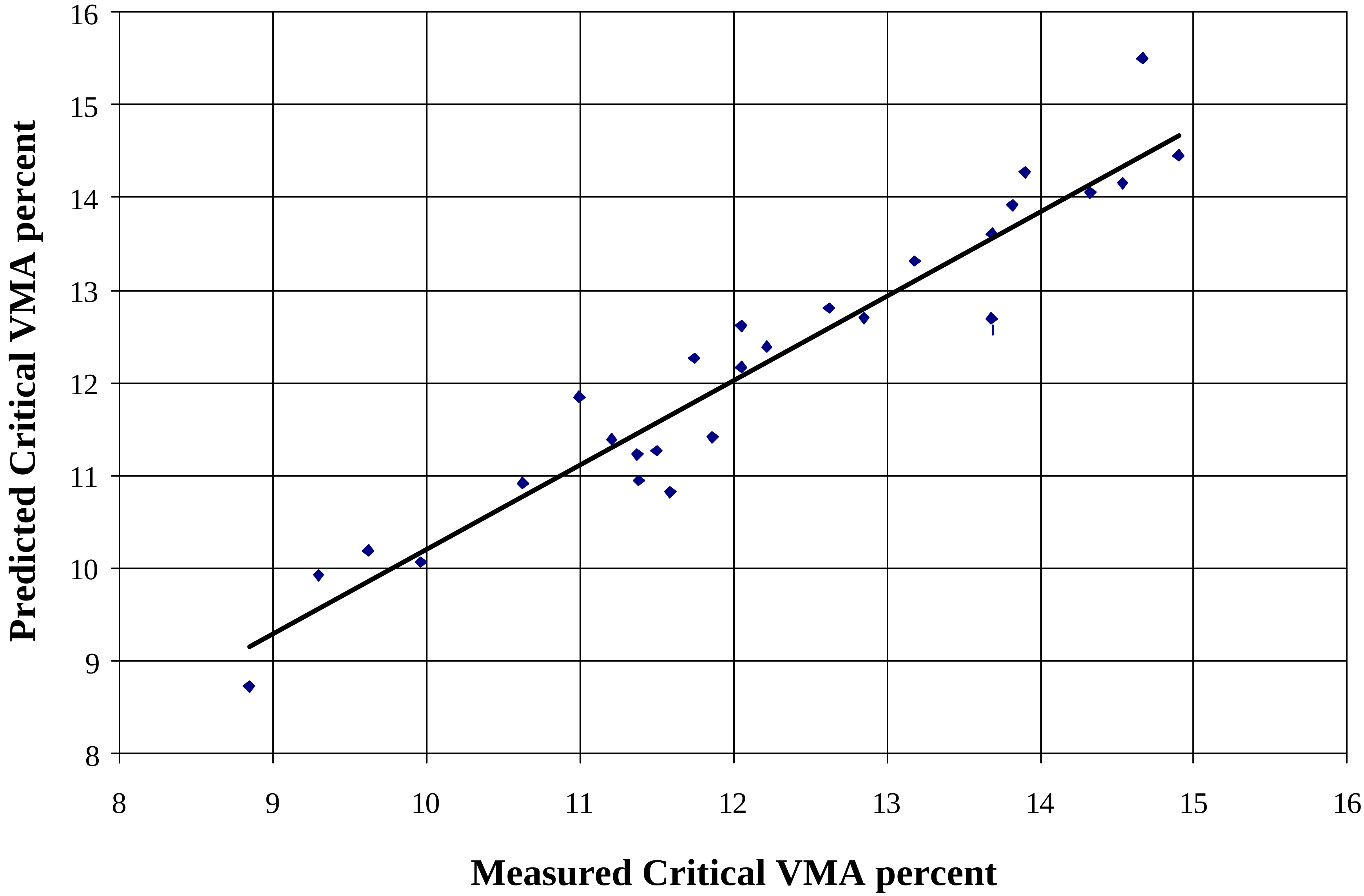

FIGURE 15 Observed Versus Predicted Critical VMA

Volume of Effective Binder

A preliminary ANOVA study similar to that undertaken for VMA was undertaken to identify the significant aggregate-related variables for the volume of effective binder (V_{be}). The same independent variables were identified, i.e., FM, CAPC, and FAPC. The resulting predictive equation is shown below:

$$V_{be} = 23.018 - 3.271FM + 0.014CAPC + 0.018FAPC \quad r^2 = 0.90 \quad see = 0.55 \quad . \tag{4}$$

The ANOVA and regression results are given in Tables 23 and 24.

TABLE 23 ANOVA V_{be} Versus FM, CAPC, FAPC and Interaction Terms

Source	Sum of Squares	Degrees of Freedom	Mean Squares
Model	77.710	27	2.858
Intercept	2528.900	1	2528.900
FM	49.935	8	6.242
CAPC	15.145	2	7.572
FAPC	9.079	1	9.079
FM x CAPC	2.301	11	0.209
FM x FAPC	0.710	5	0.142

TABLE 24 Regression V_{be} Versus FM, CAPC, and FAPC

24A Summary Output

	Regression Statistics
Multiple *R*	0.95254
R square	0.90734
Adjusted *R* square	0.89576
Standard error	0.54615
Observations	28

24B ANOVA

	df	SS	MS	*F*	Significance *F*
Regression	3	70.0970	23.3657	78.3355	0.0000
Residual	24	7.1586	0.2983		
Total	27	77.2556			

	Coefficients	Standard Error	*t*-statistic	*P*-value	Lower 95%	Upper 95%
Intercept	23.018	1.085	21.212	0.000	20.778	25.257
FM	–3.271	0.242	–13.532	0.000	–3.770	–2.772
CAPC	0.014	0.003	4.687	0.000	0.008	0.019
FAPC	0.018	0.003	6.448	0.000	0.012	0.024

The results obtained so far are shown to be significant and robust. However, it must be remembered that, as before, these critical relationships (VMA_{crit}, V_{be}) would have to be quoted together. The "older" criteria for "critical VMA" had to be quoted in conjunction with an air void content. The results derived above would equally require the accompaniment of an effective binder content. Is there any other pertinent or critical parameters that should be given?

Voids Filled with Asphalt

A parameter that has not been discussed so far is the voids filled with asphalt. This is analogous to the degree of saturation in soils, and represents the degree to which the VMA space is filled with effective binder. In the observed data, VFA is almost constant and yet is identified by ANOVA to be significantly influenced by the aggregate factors (see Table 25). The VFA results indicate that mixtures transition from sound to unsound at a value of VFA in the range 63 to 83 percent. The average value found at critical VMA is 77 percent, with a standard error of estimate of 1.09 percent.

$$VFA_{crit} = 97.3961 - 5.3343FM + 0.03187CAPC + 0.05838FAPC \quad r^2 = 0.96 \quad see = 0.70 \quad . \tag{5}$$

TABLE 25 Regression VFA Versus FM, CAPC, and FAPC

25A Summary Output

	Regression Statistics
Multiple *R*	0.98331
R square	0.96690
Adjusted *R* square	0.96277
Standard error	0.69596
Observations	28

25B ANOVA

	df	SS	MS	*F*	Significance *F*
Regression	3	339.60161	113.20054	233.70881	0.00000
Residual	24	11.62478	0.48437		
Total	27	351.22639			

	Coefficients	Standard Error	*t*-statistic	*P*-value	Lower 95%	Upper 95%
Intercept	97.39611	1.38278	70.43503	0.00000	94.54219	100.25002
FM	–5.33433	0.30808	–17.31468	0.00000	–5.97018	–4.69848
CAPC	0.03187	0.00367	8.67575	0.00000	0.02429	0.03945
FAPC	0.05838	0.00355	16.43152	0.00000	0.05105	0.06572

The predictive equations derived thus far provide a means by which the *critical* state of a mixture may be estimated, based on aggregate factors. These are not design criteria. It is still desirable to ensure a range of air voids in a laboratory compacted mixture. It is still desirable to ensure adequate coating on the aggregate particles. It is still desirable to prevent acceptance of unacceptable mixtures.

The volume percentage of effective binder, V_{be}, is relatively insensitive to the level of compaction and may be considered a reasonable design parameter. The difference between V_{be} at critical state and at 4 percent air voids is minimal (~0.13 percent). This value could be used as a design requirement.

Summary

The primary volumetric parameter must be considered to be the effective binder volume, V_{be}. This is bounded by the minimum amount of binder necessary to provide an adequate coating to the aggregate and by an amount beyond which drain-down might be observed. The Iowa DOT defines these limits using the empirical measure of film thickness. film thickness is a composite measure of effective binder volume and the normal surface area of the blended aggregate.

$$FT = \frac{10P_{be}}{SA} = \frac{10G_b V_{be}}{SA \times G_{mb}} \quad . \tag{6}$$

The effective volume of binder, V_{be}, may be determined in either of two ways:

1. By defining a desirable film thickness, FT, which will, in conjunction with a measured surface area, SA, yield an effective binder content, P_{be}. Then a desirable (target) effective binder volume may be found from $V_{be} = P_{be} \times G_{mb}/G_b$. This assumes that the bulk specific gravity of the mixture, G_{mb}, is known or can be estimated.

2. By using the regression relationship given above, in equation (4), based on the aggregate properties,

$$V_{be} = 23.018 - 3.271FM + 0.014CAPC + 0.018FAPC \quad . \tag{7}$$

Having defined a desirable binder content (volume), a critical VMA may be defined indirectly using the relationship VFA = V_{be}/VMA x 100 or VMA = V_{be}/VFA x 100. In this relationship, the magnitude of VFA is based on the aggregate factors through the regression equation (5):

$$VFA_{crit} = 97.3961 - 5.3343FM + 0.03187CAPC + 0.05838FAPC \quad . \tag{8}$$

This critical state will be found to occur at an air void content V_a = VMA – V_{be}. This will typically observed to be less than 4 percent. To translate these critically identified values to a design requirement at 4 percent air voids, it will be necessary to adjust the critical values to design values; thus,

$$VMA_{des} = 4 + \left[\frac{96V_{be}}{(100 - V_a)}\right]_{crit}$$

$$VFA_{des} = \frac{\left(VMA_{des} - 4\right)}{VMA_{des}} \times 100 \quad . \qquad (9)$$

An example of this process is given as follows: A blended aggregate with a fineness modulus of 5.0 comprising a coarse aggregate with 85 percent crushed particles is to be used with a wholly (100 percent) manufactured sand. The volume percentage of effective binder is found from equation (7) to be

$$V_{be} = 23.018 - 3.271(5.0) + 0.014(85) + 0.018(100) = 9.65 \text{ percent} \quad .$$

The VFA is estimated using equation (8) is found to be

$$VFA_{crit} = 97.3961 - 5.3343(5.0) + 0.03187(85) + 0.05838(100) = 79.27 \text{ percent} \quad .$$

The critical VMA is found to be

$$VMA_{crit} = V_{be}/VFA_{crit} \times 100 = 9.65/79.27 \times 100 = 12.17 \text{ percent} \quad .$$

Similar results may be obtained using equation (3), i.e.,

$$VMA_{crit} = 26.20 - 3.34 \times 5.0 + 0.0129 \times 85 + 0.0155 \times 100 = 12.15 \text{ percent} \quad .$$

However, use of the latter relationship precludes the discretionary selection of a desirable (or target) film thickness. The critical air void content is

$$V_a = VMA_{crit} - V_{be} = 12.17 - 9.65 = 2.52 \text{ percent} \quad .$$

For design purposes (at V_a = 4 percent), the design VMA (at V_a = 4 percent) is found to be

$$VMA_{des} = 4 + (96 \times 9.65)/(100 - 2.52) = 13.50 \text{ percent} \quad .$$

The mixture should be sound at this volumetric state and should remain sound until the air voids are reduced to 2.5 percent and a VMA of 12.2 percent.

General Discussion of Critical State in HMA

The above analysis was entirely predicated on a specific compaction energy appropriate to 109 gyrations of an SGC. The question remains, What about other levels of compaction? This question leads to an interesting discussion on asphalt compaction and mixture soundness.

If we draw on the experience of the soils engineering fraternity, we can find an analogous technology in the compaction of soil materials and moisture-density relationships. A soil material is also a ternary material (soil-water-air) and is therefore generally an analog to hot-mix asphalt (aggregate-binder-air).

In the "design" of earthen embankments and subgrade preparation, soil materials are typically conditioned over a range of moisture contents and are compacted to at least one level of compaction. In research applications, more than one level of compaction may be applied. The typical results of such an exercise would appear similar to those shown in Figure 16.

The axes on this chart differ from those that are conventionally used in a HMA Marshall design. Molding water content (*m* percent, or *w* percent) is typically reported as percent by mass of solids, while binder content, P_b, is reported as percent by mass of mixture. Dry density (γ_d) indicates the ratio between the mass of the *dry* solid soil material and the total (wet) volume of the soil sample, while the unit weight (or bulk density, G_{mb}) of HMA mixtures indicates the ratio of the *total* mass of the mixture to the total volume of the HMA sample. These customary definitions are due to the separate development of the two technologies and are not materially different since either set of definitions can be mapped into the other on a one-to-one basis, as shown in Table 26.

TABLE 26 Comparison of Soil Mechanics Versus Asphalt Technology Terminology

	Soil Mechanics	Asphalt Technology	Conversion
Moisture (binder) content	W	P_b	$w = 100 \times P_b /(100 - P_b)$
Dry (bulk) density	γ_d	G_{mb}	$\gamma_d = G_{mb} (100 - P_b)/100$

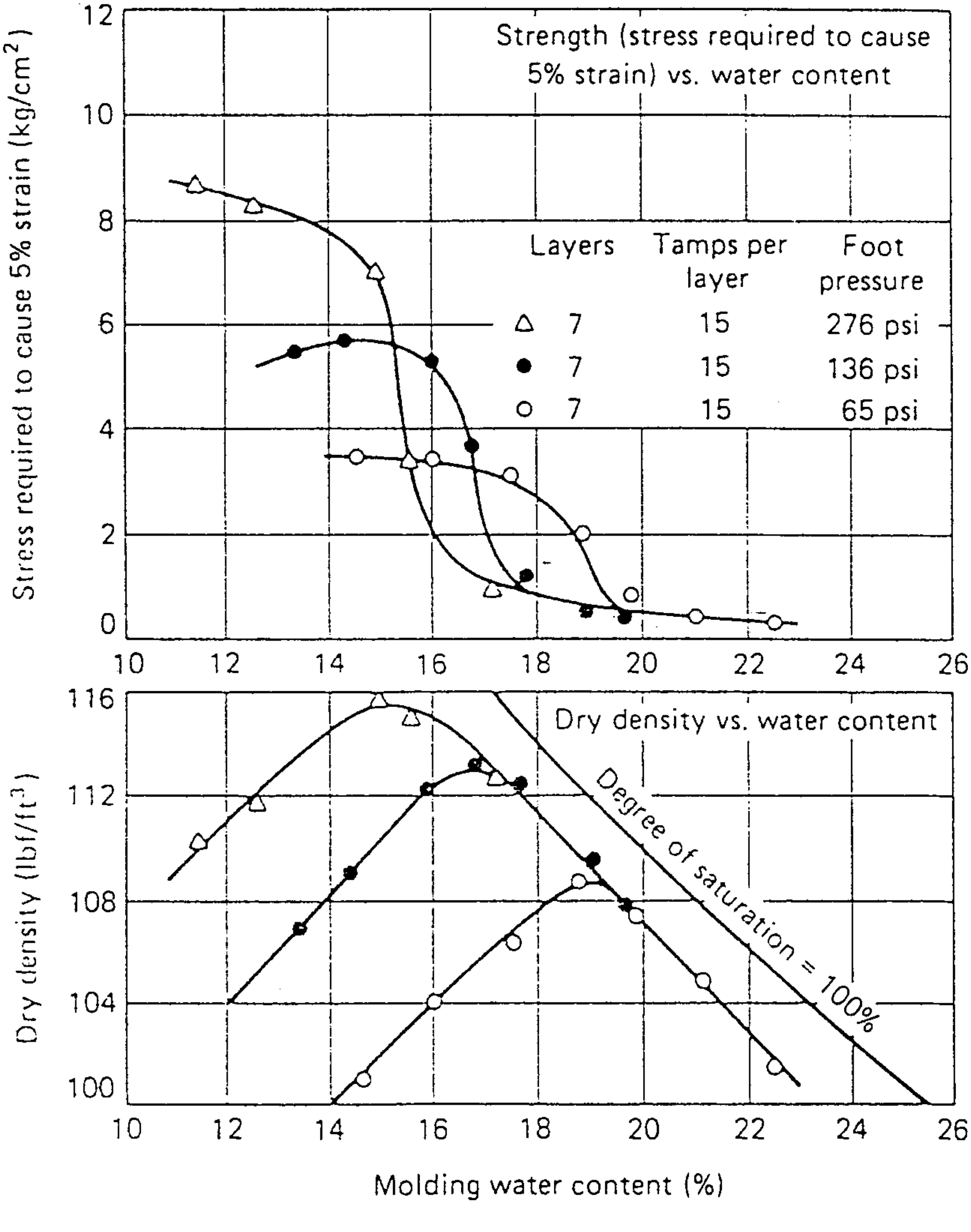

FIGURE 16 Dry Density as a Function of Water Content for Soils (*43*)

The difference between dry density (soil) and bulk density (HMA) is instructive. Translating the soil mechanics dry density into “asphaltese,” it becomes the aggregate concentration in the mixture, i.e., the mass of aggregate in the compacted mixture volume—as distinct from the more conventional bulk density—that indicates the mass of both aggregate and binder in the compacted mixture volume. The dry density of an asphalt mixture (aggregate concentration), denoted hereafter G_{md}, is related to VMA in the following manner:

$$VMA = 100 - \frac{(100 - P_b)G_{mb}}{G_{sb}} = 100 - \frac{100G_{md}}{G_{sb}} = 100\left(1 - \frac{G_{md}}{G_{sb}}\right) \quad . \tag{10}$$

From this relationship, it can be deduced that *minimum* VMA corresponds to a *maximum* aggregate concentration. Since the maximum aggregate concentration, G_{md}, occurs at a binder content less than that of maximum *density*, this explains why maximum stability in the Marshall design typically occurs at a binder content somewhat less than the binder content necessary for maximum density.

The difference between peak *dry* density (aggregate concentration) and peak density is instructive. In Figure 17, two curves are drawn, one for G_{mb} and the other for G_{md}. For discussion purposes, the graph has been divided into three phases, 1 (to the left of peak G_{md}), 2 (between peak G_{md} and peak G_{mb}), and 3 (to the right of peak G_{mb}).

> Phase 1—This phase represents a “dry” or “lean” soil or asphalt mixture. The water (binder) is insufficient to adequately lubricate the particles into a denser configuration. The properties (density) of the mixture are controlled by the friction between the particles and it exhibits a low cohesion and high friction angle.
>
> Phase 2—In this phase, the volume of the moisture (binder) exceeds that necessary to mobilize maximum friction (at peak G_{md}). Adding more moisture (binder) lubricates the aggregate particles sufficiently to overcome interparticle friction and collapse the aggregate skeleton into a more dense configuration.
>
> Phase 3—Here, the densest aggregate density has been achieved, and to insert more moisture (binder) it is necessary to *displace* some of the aggregate. This reduces the aggregate concentration and the mixture density simultaneously. The interparticle friction is reducing while cohesion is increasing.

In soil mechanics the peak dry density is identified with peak strength, as shown in Figure 16 (*43*). Likewise, for HMA, the condition at which the strength starts to drop (catastrophically) can be identified with peak aggregate concentration (*1*). In this project, the binder contents at which strain in the NAT was identified as starting to increase significantly is closely tied to the binder content at which peak aggregate concentration occurred.

In identifying the transition point from NAT data, the project team estimated (by eye) the point at which strain started to increase, interpolating between binder contents at 1 percent increments. A further estimate was made using the peak “dry density” or maximum aggregate concentration

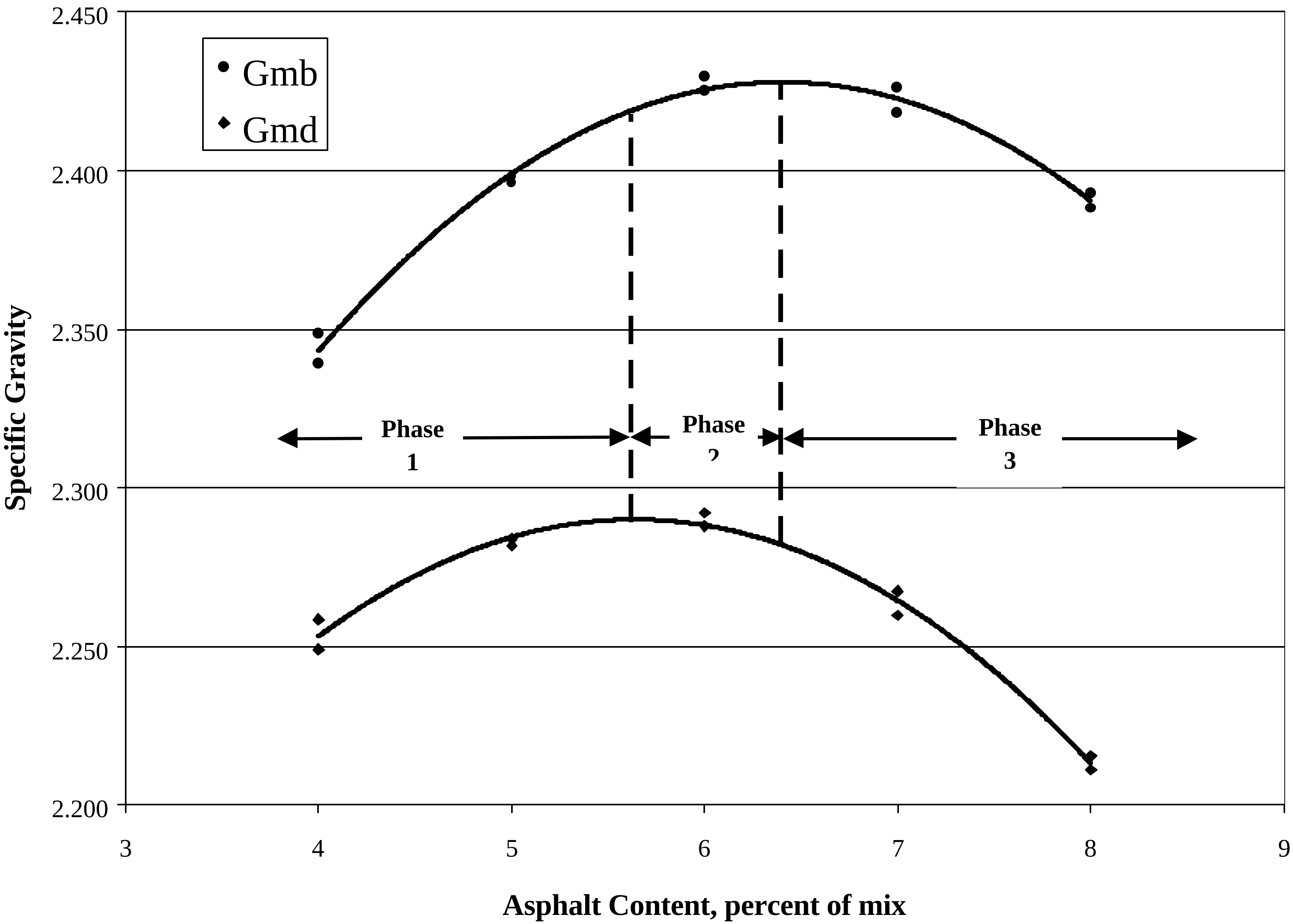

FIGURE 17 Three Phases of Asphalt-Aggregate Mixtures

(independently of the strain results). The two methods agreed remarkably well with a standard error of estimate of the differences between the two methods of 0.06 percent. We believe that this parameter (i.e., maximum aggregate concentration) provides a powerful method of identifying a *critical* state in asphalt mixtures.

Having just identified the maximum aggregate concentration as a robust identifier of a critical state transition in HMA mixtures, it must be recalled that *maximum aggregate concentration* corresponds to a condition of *minimum VMA*. Thus it must be concluded that the *minimum* VMA is the *critical* VMA, where the term “minimum VMA” is defined to indicate the smallest VMA measured on a given aggregate blend when compacted with a given energy over a range of binder contents, and critical VMA represents a VMA threshold between sound and unsound performance.

The practical implication of this conclusion is that McLeod/Superpave minimum VMA specifications are restrictive and unnecessary. Sound mixtures can be specified to have a sufficient coating of binder (V_{be}) and not to exceed a proven saturation of binder in the VMA space (VFA)—as Bruce Marshall originally proposed and the United States Army COE continues to specify—without a minimum VMA requirement. It is believed that this concept, in conjunction with the methodology proposed above, will provide the necessary and sufficient conditions to specify and design SUperior PERforming Asphalt PAVEments.

6 CONCLUSIONS AND RECOMMENDATIONS

Conclusions

Mix designers tend to target mixture parameters close to the critical VMA, even for mixtures that might not be dense graded. It appears prudent to expand and refine the relationship to include the effects of aggregate-related factors such as gradation, percent crushed coarse aggregate, and percent crushed fine aggregate. The goals of this project were to examine whether or not this was feasible and, if so, to provide a rational method for adjusting the current minimum VMA–nominal maximum aggregate size relationship. It must be emphasized that the conclusions are based upon carefully controlled laboratory testing of a limited number of specimens and have not been verified in the field.

Based on the literature search, laboratory testing, and analysis of test data, the following conclusions are made:

Literature Review

1. The definition of minimum (or critical) VMA adopted by Superpave is dependent only upon nominal maximum aggregate size without regard to other significant aggregate-related properties (*35*).

2. The minimum VMA criterion adopted by the SHRP Expert Task Group for Superpave was essentially that proposed by Norman McLeod in 1959 (*2*).

3. The available literature on the development of the minimum VMA criterion is sketchy; McLeod presented his relationship without the research or data from which it derived and suggested that it would be modified with experience and test data (*2*).

4. The implementation of Superpave has brought significant awareness of and renewed focus on how difficult and problematic meeting the minimum VMA criterion can be for mix designers (*3, 4*).

5. Prior to SHRP, there was some awareness of difficulties in meeting minimum VMA. Some researchers attempted to develop rational methods of increasing VMA based on gradation, and others modified the criterion to account for gradation. (*19, 27*).

6. There is considerable interest in using asphalt film thickness either to supplement or to replace the minimum VMA criteria (*3, 5, 6*).

7. The laboratory tests that seem best suited for determining the critical state transition of asphalt paving mixtures are the permanent deformation tests. Reviewing the literature, there is not a consensus as to which laboratory test would best distinguish the critical state of VMA. Based on cost, availability, ease of use, and the SHRP findings (*9*), the repeated load triaxial test apparatus appears to be the preferred method.

8. Several researchers have pointed out aggregate factors other than nominal maximum aggregate size that affect VMA. These include percent filler, shape, surface texture, percent crushed aggregate, fine aggregate angularity, and coarseness of the gradation.

Analysis of Test Data

1. As shown in Figure 13, the specified VMA values provided by Superpave (Table 17) do not appear to be adequate for identifying mixture performance; only three out of 28 results were correctly identified, a success rate of about 11 percent. The three “correctly” identified mixtures still have the potential to become unstable while meeting the specified VMA values.

2. The volume percentage of effective binder, V_{be}, is relatively insensitive to the level of compaction and appears to be a critical parameter. As shown in Figure 14, the reliability of a V_{be} criterion is 10 out of 28.

3. ANOVA analysis of the test data identified three factors—fineness modulus (FM) , coarse aggregate percent crushed (CAPC), and fine aggregate percent crushed (FAPC) and two interactions (FM x CAPC and FM x FAPC)—as significant.

4. ANOVA analysis identified the nominal maximum aggregate size (NMAS) and surface area (SA) of the gradation as being of no statistical significance when the fineness modulus was included in the analysis.

5. Linear regression analysis showed the current VMA specification (VMA versus log[NMAS]) had an adjusted r^2 value of 0.47.

6. Linear regression analysis of VMA versus FM, CAPC, and FAPC had an adjusted r^2 value of 0.88.

7. Linear regression analysis of V_{be} versus FM, CAPC, and FAPC had an adjusted r^2 value of 0.90.

8. Linear regression analysis of VFA versus FM, CAPC, and FAPC had an adjusted r^2 value of 0.96.

9. The maximum aggregate concentration (minimum VMA) appears to be a robust indicator of the critical state transition in asphalt paving mixtures.

Summary

Thus from the literature review, testing, and statistical analysis performed on this project, it appears that the current minimum VMA requirements specified in Superpave mix design protocol are overly restrictive and unnecessary, ruling out candidate aggregate gradations that should perform adequately.

Two factors clearly stand out that differentiate sound from unsound mixtures are: a sufficient coating of binder (V_{be}) and not overly saturating the VMA with binder (VFA).

Recommendations

The literature review, testing, and statistical analysis performed on this project have suggested the following recommendations:

1. The predictive relationships obtained in this study need to be compared with field data and verified or adjusted as necessary.

2. In place of the current minimum VMA specification, a durability criterion based on the more robust parameters of VFA or V_{be} should be used in designing asphalt mixtures.

3. If a minimum VMA is to be specified, it should include fineness modulus, coarse aggregate percent crushed, fine aggregate percent crushed, and their interactions.

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APPENDIX A

VOLUMETRIC DATA RESULTS

Tables A.1–A.4 use the following property definitions:

G_{sb} =	bulk specific gravity of the aggregate,
SA =	surface area,
G_{se} =	effective specific gravity of the aggregate,
Abs. (%) =	percent asphalt absorbtion,
G_{mb} =	bulk specific gravity of the compacted HMA specimen,
G_{mm} =	theoretical maximum specific gravity of the HMA,
Air voids =	percent air voids in the compacted HMA specimen,
VMA =	voids in the mineral aggregate,
VFA =	voids filled with asphalt,
Dust-P_{be} ratio =	ratio of P200 material to effective asphalt content,
K =	richness modulus, and
Film thickness =	average asphalt film thickness (microns).

Table A.1 Summary of Volumetric Results for 100 Percent Crushed Specimens

Property	9.5 mm			12.5 mm			19 mm		
	F	D	C	F	D	C	F	D	C
G_{sb}	2.647	2.628	2.612	2.631	2.616	2.604	2.624	2.608	2.599
SA	6.65	5.95	4.98	5.99	5.37	4.68	5.80	5.02	4.56
G_{se}	2.736	2.727	2.724	2.728	2.727	2.718	2.727	2.722	2.721
Abs. (%)	1.26	1.39	1.61	1.39	1.59	1.65	1.52	1.64	1.76

G_{mb}	9.5 mm			12.5 mm			19 mm		
	F	D	C	F	D	C	F	D	C
4	2.213	2.292	2.310	2.275	2.325	2.353	2.307	2.358	2.362
4	2.199	2.291	2.304	2.285	2.336	2.352	2.301	2.352	2.390
5	2.236	2.319	2.363	2.318	2.367	2.390	2.350	2.402	2.415
5	2.220	2.347	2.341	2.305	2.369	2.396	2.345	2.397	2.402
6	2.256	2.350	2.381	2.337	2.400	2.438	2.360	2.446	2.438
6	2.282	2.355	2.412	2.331	2.405	2.435	2.359	2.447	2.442
7	2.289	2.414	2.413	2.376	2.435	2.429	2.411	2.432	2.433
7	2.308	2.410	2.413	2.396	2.433	2.425	2.417	2.432	2.431
8	2.351	2.403	2.408	2.400	2.409	2.406	2.406	2.409	2.403
8	2.336	2.403	2.405	2.400	2.405	2.404	2.406	2.400	2.409

G_{mm}	9.5 mm			12.5 mm			19 mm		
	F	D	C	F	D	C	F	D	C
4	2.564	2.557	2.553	2.556	2.559	2.551	2.558	2.552	2.553
4	2.564	2.557	2.553	2.556	2.559	2.551	2.558	2.552	2.553
5	2.525	2.518	2.514	2.516	2.521	2.512	2.518	2.513	2.513
5	2.525	2.518	2.514	2.516	2.521	2.512	2.518	2.513	2.513
6	2.486	2.480	2.476	2.477	2.483	2.475	2.480	2.475	2.475
6	2.486	2.480	2.476	2.477	2.483	2.475	2.480	2.475	2.475
7	2.449	2.443	2.439	2.440	2.446	2.439	2.444	2.438	2.438
7	2.449	2.443	2.439	2.440	2.446	2.439	2.444	2.438	2.438
8	2.413	2.407	2.403	2.404	2.411	2.404	2.408	2.402	2.403
8	2.413	2.407	2.403	2.404	2.411	2.404	2.408	2.402	2.403

Air Voids	9.5 mm			12.5 mm			19 mm		
	F	D	C	F	D	C	F	D	C
4	13.7%	10.4%	9.5%	11.0%	10.4%	7.8%	9.9%	8.2%	7.5%
4	14.3%	10.4%	9.8%	10.6%	10.5%	7.8%	10.1%	8.4%	6.4%
5	11.4%	7.9%	6.0%	7.9%	8.0%	4.9%	6.7%	5.2%	3.9%
5	12.1%	6.8%	6.9%	8.4%	6.9%	4.6%	6.9%	5.4%	4.4%
6	9.2%	5.3%	3.8%	5.6%	5.4%	1.5%	4.9%	2.1%	1.5%
6	8.2%	5.1%	2.6%	5.9%	5.2%	1.6%	4.9%	2.0%	1.3%
7	6.5%	1.2%	1.0%	2.6%	1.3%	0.4%	1.3%	1.3%	0.2%
7	5.8%	1.4%	1.1%	1.8%	1.5%	0.6%	1.1%	1.3%	0.3%
8	2.5%	0.2%	-0.2%	0.2%	0.3%	-0.1%	0.1%	0.9%	0.0%
8	3.2%	0.2%	-0.1%	0.1%	0.3%	0.0%	0.1%	1.3%	-0.2%

Table A.1 ***Continued***

VMA	9.5 mm			12.5 mm			19 mm		
	F	D	C	F	D	C	F	D	C
4	19.7%	16.3%	15.1%	17.0%	15.9%	13.2%	15.6%	13.2%	12.7%
4	20.3%	16.3%	15.3%	16.6%	15.9%	13.3%	15.8%	13.4%	11.7%
5	19.8%	16.2%	14.1%	16.3%	15.8%	12.8%	14.9%	12.5%	11.7%
5	20.3%	15.1%	14.9%	16.7%	14.8%	12.6%	15.1%	12.7%	12.2%
6	19.9%	15.9%	14.3%	16.5%	15.6%	12.0%	15.4%	11.8%	11.8%
6	19.0%	15.8%	13.2%	16.7%	15.4%	12.1%	15.5%	11.8%	11.7%
7	19.6%	14.6%	14.1%	16.0%	14.2%	13.2%	14.5%	13.3%	12.9%
7	18.9%	14.7%	14.1%	15.3%	14.3%	13.4%	14.3%	13.3%	13.0%
8	18.3%	15.9%	15.2%	16.1%	15.5%	15.0%	15.6%	15.0%	14.9%
8	18.8%	15.9%	15.3%	16.1%	15.5%	15.1%	15.6%	15.3%	14.7%

VFA	9.5 mm			12.5 mm			19 mm		
	F	D	C	F	D	C	F	D	C
4	30.6%	36.2%	36.9%	35.3%	34.3%	41.4%	36.8%	37.9%	41.5%
4	29.6%	36.1%	36.3%	36.3%	34.2%	41.2%	36.3%	37.2%	45.6%
5	42.1%	51.1%	57.2%	51.7%	49.4%	62.0%	54.9%	58.7%	66.7%
5	40.6%	55.2%	53.7%	50.0%	53.5%	63.2%	54.1%	57.7%	63.6%
6	53.5%	67.0%	73.3%	65.7%	65.6%	87.3%	68.5%	82.3%	87.3%
6	56.6%	67.9%	80.4%	64.6%	66.5%	86.5%	68.3%	82.7%	88.5%
7	66.6%	91.9%	92.6%	83.7%	90.8%	97.1%	90.8%	90.2%	98.3%
7	69.6%	90.8%	92.4%	88.1%	89.7%	95.7%	92.3%	90.0%	97.6%
8	86.1%	98.8%	101.4%	99.0%	97.9%	100.6%	99.5%	93.9%	100.1%
8	83.1%	98.9%	100.5%	99.1%	98.0%	100.1%	99.6%	91.6%	101.7%

Dust-P_{be} Ratio	9.5 mm			12.5 mm			19 mm		
	F	D	C	F	D	C	F	D	C
4	1.5	1.5	1.6	1.5	1.6	1.7	1.6	1.7	1.8
4	1.5	1.5	1.6	1.5	1.6	1.7	1.6	1.7	1.8
5	1.1	1.1	1.2	1.1	1.2	1.2	1.1	1.2	1.2
5	1.1	1.1	1.2	1.1	1.2	1.2	1.1	1.2	1.2
6	0.8	0.9	0.9	0.9	0.9	0.9	0.9	0.9	0.9
6	0.8	0.9	0.9	0.9	0.9	0.9	0.9	0.9	0.9
7	0.7	0.7	0.7	0.7	0.7	0.7	0.7	0.7	0.8
7	0.7	0.7	0.7	0.7	0.7	0.7	0.7	0.7	0.8
8	0.6	0.6	0.6	0.6	0.6	0.6	0.6	0.6	0.6
8	0.6	0.6	0.6	0.6	0.6	0.6	0.6	0.6	0.6

K	9.5 mm			12.5 mm			19 mm		
	F	D	C	F	D	C	F	D	C
4	2.58	2.54	2.48	2.53	2.48	2.43	2.51	2.45	2.41
5	3.26	3.21	3.13	3.20	3.14	3.07	3.18	3.10	3.05
6	3.96	3.89	3.80	3.88	3.81	3.72	3.85	3.76	3.70
7	4.66	4.59	4.48	4.57	4.49	4.39	4.54	4.43	4.36
8	5.39	5.30	5.18	5.28	5.18	5.07	5.25	5.12	5.04

Table A.1 ***Continued***

Film Thickness	9.5 mm			12.5 mm			19 mm		
	F	**D**	**C**	**F**	**D**	**C**	**F**	**D**	**C**
4	4.1	4.4	4.8	4.4	4.5	5.0	4.3	4.7	4.9
5	5.6	6.1	6.8	6.0	6.4	7.2	6.1	6.7	7.1
6	7.1	7.7	8.8	7.7	8.2	9.3	7.8	8.7	9.3
7	8.6	9.4	10.8	9.4	10.1	11.4	9.5	10.7	11.5
8	10.1	11.1	12.8	11.0	11.9	13.6	11.2	12.7	13.7

Table A.2 Volumetric Results for 50 Percent Crushed/ 50 Percent Natural Specimens

Property	9.5 mm			12.5 mm			19 mm		
	F	D	C	F	D	C	F	D	C
G_{sb}	2.613	2.593	2.577	2.597	2.585	2.573	2.592	2.578	2.571
SA	6.65	5.95	4.98	5.99	5.37	4.68	5.80	5.02	4.56
G_{se}	2.696	0.000	2.691	37.318	37.257	37.116	37.189	37.032	37.065
Abs. (%)	1.21	1.30	1.69	1.21	1.46	1.78	1.43	1.79	1.88

G_{mb}	9.5 mm			12.5 mm			19 mm		
	F	D	C	F	D	C	F	D	C
4	2.215	2.273	2.295	2.294	2.340	2.351	2.386	2.389	2.427
4	2.198	2.272	2.279	2.312	2.346	2.360	2.393	2.399	2.390
5	2.245	2.309	2.316	2.346	2.392	2.386	2.422	2.408	2.448
5	2.224	2.323	2.321	2.329	2.395	2.396	2.440	2.399	2.426
6	2.277	2.369	2.365	2.356	2.427	2.425	2.436	2.423	2.449
6	2.284	2.343	2.362	2.380	2.437	2.427	2.444	2.435	2.439
7	2.305	2.388	2.402	2.394	2.418	2.415	2.424	2.411	2.395
7	2.314	2.381	2.402	2.385	2.412	2.409	2.421	2.385	2.416
8	2.347	2.374	2.380	2.380	2.380	2.381	2.398		
8	2.346	2.381	2.378	2.382	2.376	2.384	2.390	2.376	2.357

G_{mm}	9.5 mm			12.5 mm			19 mm		
	F	D	C	F	D	C	F	D	C
4	2.529	2.528	2.527	2.519	2.517	2.531	2.528	2.535	2.535
4	2.529	2.528	2.527	2.519	2.517	2.531	2.528	2.535	2.535
5	2.491	2.493	2.489	2.481	2.478	2.493	2.491	2.497	2.497
5	2.491	2.493	2.489	2.481	2.478	2.493	2.491	2.497	2.497
6	2.453	2.458	2.453	2.445	2.441	2.456	2.455	2.459	2.461
6	2.453	2.458	2.453	2.445	2.441	2.456	2.455	2.459	2.461
7	2.417	2.424	2.417	2.410	2.405	2.421	2.419	2.423	2.425
7	2.417	2.424	2.417	2.410	2.405	2.421	2.419	2.423	2.425
8	2.382	2.392	2.382	2.376	2.369	2.386	2.385		
8	2.382	2.392	2.382	2.376	2.369	2.386	2.385	2.388	2.391

Air Voids	9.5 mm			12.5 mm			19 mm		
	F	D	C	F	D	C	F	D	C
4	12.4%	10.1%	9.2%	8.9%	7.0%	7.1%	5.6%	5.7%	4.3%
4	13.1%	10.1%	9.8%	8.2%	6.8%	6.8%	5.3%	5.3%	5.7%
5	9.9%	7.3%	7.0%	5.5%	3.5%	4.3%	2.8%	3.5%	2.0%
5	10.7%	6.8%	6.8%	6.1%	3.4%	3.9%	2.0%	3.9%	2.9%
6	7.2%	3.6%	3.6%	3.6%	0.6%	1.3%	0.7%	1.5%	0.5%
6	6.9%	4.7%	3.7%	2.7%	0.1%	1.2%	0.4%	1.0%	0.9%
7	4.6%	1.5%	0.6%	0.7%	-0.5%	0.2%	-0.2%	0.5%	1.2%
7	4.3%	1.8%	0.6%	1.1%	-0.3%	0.5%	-0.1%	1.6%	0.4%
8	1.4%	0.8%	0.1%	-0.2%	-0.5%	0.2%	-0.5%		
8	1.5%	0.5%	0.2%	-0.3%	-0.3%	0.1%	-0.2%	0.5%	1.4%

Table A.2 *Continued*

VMA	9.5 mm			12.5 mm			19 mm		
	F	D	C	F	D	C	F	D	C
4	18.6%	15.8%	14.5%	15.2%	13.1%	12.3%	11.6%	11.0%	9.4%
4	19.2%	15.9%	15.1%	14.6%	12.9%	12.0%	11.4%	10.7%	10.8%
5	18.4%	15.4%	14.6%	14.2%	12.1%	11.9%	11.2%	11.3%	9.5%
5	19.1%	14.9%	14.4%	14.8%	12.0%	11.5%	10.5%	11.6%	10.3%
6	18.1%	14.1%	13.7%	14.7%	11.7%	11.4%	11.6%	11.7%	10.5%
6	17.8%	15.0%	13.8%	13.9%	11.4%	11.3%	11.4%	11.2%	10.8%
7	18.0%	14.4%	13.3%	14.3%	13.0%	12.7%	13.0%	13.0%	13.3%
7	17.7%	14.6%	13.3%	14.6%	13.2%	12.9%	13.1%	14.0%	12.6%
8	17.4%	15.8%	15.0%	15.7%	15.3%	14.9%	14.9%		
8	17.4%	15.5%	15.1%	15.6%	15.4%	14.8%	15.2%	15.2%	15.7%

VFA	9.5 mm			12.5 mm			19 mm		
	F	D	C	F	D	C	F	D	C
4	33.3%	36.3%	36.6%	41.3%	46.4%	42.2%	51.7%	48.0%	54.4%
4	32.0%	36.2%	34.9%	43.5%	47.2%	43.5%	52.9%	49.9%	46.7%
5	46.4%	52.2%	52.3%	61.6%	71.1%	63.9%	75.4%	68.5%	79.2%
5	44.1%	54.3%	53.1%	58.6%	71.9%	66.4%	80.9%	66.3%	72.3%
6	60.3%	74.4%	74.0%	75.3%	95.2%	88.9%	93.6%	87.3%	95.4%
6	61.4%	69.0%	73.2%	80.7%	98.7%	89.6%	96.1%	91.3%	92.0%
7	74.2%	89.5%	95.3%	95.4%	104.2%	98.1%	101.4%	96.1%	90.7%
7	75.8%	87.7%	95.3%	92.8%	102.4%	96.4%	100.6%	88.6%	97.0%
8	91.7%	95.2%	99.4%	101.0%	103.0%	98.7%	103.5%		
8	91.4%	97.1%	98.8%	101.7%	101.8%	99.3%	101.3%	96.6%	90.9%

Dust-P_{be} Ratio	9.5 mm			12.5 mm			19 mm		
	F	D	C	F	D	C	F	D	C
4	1.41	1.47	1.71	1.40	1.56	1.78	1.52	1.75	1.82
4	1.41	1.47	1.71	1.40	1.56	1.78	1.52	1.75	1.82
5	1.04	1.08	1.20	1.04	1.12	1.23	1.10	1.21	1.24
5	1.04	1.08	1.20	1.04	1.12	1.23	1.10	1.21	1.24
6	0.82	0.85	0.92	0.82	0.88	0.94	0.86	0.93	0.94
6	0.82	0.85	0.92	0.82	0.88	0.94	0.86	0.93	0.94
7	0.68	0.70	0.75	0.68	0.72	0.76	0.71	0.75	0.76
7	0.68	0.70	0.75	0.68	0.72	0.76	0.71	0.75	0.76
8	0.58	0.60	0.63	0.58	0.61	0.64	0.60	0.63	0.64
8	0.58	0.60	0.63	0.58	0.61	0.64	0.60	0.63	0.64

K	9.5 mm			12.5 mm			19 mm		
	F	D	C	F	D	C	F	D	C
4	2.55	2.51	2.45	2.50	2.45	2.40	2.48	2.43	2.39
5	3.22	3.17	3.09	3.16	3.10	3.03	3.14	3.06	3.02
6	3.91	3.84	3.75	3.83	3.76	3.68	3.81	3.72	3.66
7	4.61	4.53	4.42	4.51	4.43	4.34	4.49	4.38	4.31
8	5.32	5.23	5.11	5.22	5.12	5.01	5.18	5.06	4.98

Table A.2 ***Continued***

Film Thickness	9.5 mm			12.5 mm			19 mm		
	F	D	C	F	D	C	F	D	C
4	4.2	4.5	4.6	4.7	4.7	4.7	4.4	4.4	4.7
5	5.7	6.2	6.6	6.3	6.6	6.9	6.4	6.4	6.8
6	7.2	7.9	8.7	8.0	8.5	9.0	8.4	8.4	9.0
7	8.7	9.6	10.7	9.7	10.3	11.1	10.4	10.4	11.2
8	10.2	11.3	12.7	11.3	12.2	13.3	11.3	12.4	13.4

Table A.3 Volumetric Results for Manufactured Fine-Natural Coarse Specimens

Property	9.5 mm			12.5 mm			19 mm		
	F	D	C	F	D	C	F	D	C
G_{sb}	2.613	2.593	2.577	2.597	2.585	2.573	2.592	2.578	2.571
SA	6.65	5.95	4.98	5.99	5.37	4.68	5.80	5.02	4.56
G_{se}	2.696	0.000	2.691	37.318	37.257	37.116	37.189	37.032	37.065
Abs. (%)	1.21	1.30	1.69	1.21	1.46	1.78	1.43	1.79	1.88

G_{mb}	9.5 mm			12.5 mm			19 mm		
	F	D	C	F	D	C	F	D	C
4	2.215	2.273	2.295	2.294	2.340	2.351	2.386	2.389	2.427
4	2.198	2.272	2.279	2.312	2.346	2.360	2.393	2.399	2.390
5	2.245	2.309	2.316	2.346	2.392	2.386	2.422	2.408	2.448
5	2.224	2.323	2.321	2.329	2.395	2.396	2.440	2.399	2.426
6	2.277	2.369	2.365	2.356	2.427	2.425	2.436	2.423	2.449
6	2.284	2.343	2.362	2.380	2.437	2.427	2.444	2.435	2.439
7	2.305	2.388	2.402	2.394	2.418	2.415	2.424	2.411	2.395
7	2.314	2.381	2.402	2.385	2.412	2.409	2.421	2.385	2.416
8	2.347	2.374	2.380	2.380	2.380	2.381	2.398		
8	2.346	2.381	2.378	2.382	2.376	2.384	2.390	2.376	2.357

G_{mm}	9.5 mm			12.5 mm			19 mm		
	F	D	C	F	D	C	F	D	C
4	2.529	2.528	2.527	2.519	2.517	2.531	2.528	2.535	2.535
4	2.529	2.528	2.527	2.519	2.517	2.531	2.528	2.535	2.535
5	2.491	2.493	2.489	2.481	2.478	2.493	2.491	2.497	2.497
5	2.491	2.493	2.489	2.481	2.478	2.493	2.491	2.497	2.497
6	2.453	2.458	2.453	2.445	2.441	2.456	2.455	2.459	2.461
6	2.453	2.458	2.453	2.445	2.441	2.456	2.455	2.459	2.461
7	2.417	2.424	2.417	2.410	2.405	2.421	2.419	2.423	2.425
7	2.417	2.424	2.417	2.410	2.405	2.421	2.419	2.423	2.425
8	2.382	2.392	2.382	2.376	2.369	2.386	2.385		
8	2.382	2.392	2.382	2.376	2.369	2.386	2.385	2.388	2.391

Air Voids	9.5 mm			12.5 mm			19 mm		
	F	D	C	F	D	C	F	D	C
4	12.4%	10.1%	9.2%	8.9%	7.0%	7.1%	5.6%	5.7%	4.3%
4	13.1%	10.1%	9.8%	8.2%	6.8%	6.8%	5.3%	5.3%	5.7%
5	9.9%	7.3%	7.0%	5.5%	3.5%	4.3%	2.8%	3.5%	2.0%
5	10.7%	6.8%	6.8%	6.1%	3.4%	3.9%	2.0%	3.9%	2.9%
6	7.2%	3.6%	3.6%	3.6%	0.6%	1.3%	0.7%	1.5%	0.5%
6	6.9%	4.7%	3.7%	2.7%	0.1%	1.2%	0.4%	1.0%	0.9%
7	4.6%	1.5%	0.6%	0.7%	-0.5%	0.2%	-0.2%	0.5%	1.2%
7	4.3%	1.8%	0.6%	1.1%	-0.3%	0.5%	-0.1%	1.6%	0.4%
8	1.4%	0.8%	0.1%	-0.2%	-0.5%	0.2%	-0.5%		
8	1.5%	0.5%	0.2%	-0.3%	-0.3%	0.1%	-0.2%	0.5%	1.4%

Table A.3 *Continued*

VMA	9.5 mm			12.5 mm			19 mm		
	F	D	C	F	D	C	F	D	C
4	18.6%	15.8%	14.5%	15.2%	13.1%	12.3%	11.6%	11.0%	9.4%
4	19.2%	15.9%	15.1%	14.6%	12.9%	12.0%	11.4%	10.7%	10.8%
5	18.4%	15.4%	14.6%	14.2%	12.1%	11.9%	11.2%	11.3%	9.5%
5	19.1%	14.9%	14.4%	14.8%	12.0%	11.5%	10.5%	11.6%	10.3%
6	18.1%	14.1%	13.7%	14.7%	11.7%	11.4%	11.6%	11.7%	10.5%
6	17.8%	15.0%	13.8%	13.9%	11.4%	11.3%	11.4%	11.2%	10.8%
7	18.0%	14.4%	13.3%	14.3%	13.0%	12.7%	13.0%	13.0%	13.3%
7	17.7%	14.6%	13.3%	14.6%	13.2%	12.9%	13.1%	14.0%	12.6%
8	17.4%	15.8%	15.0%	15.7%	15.3%	14.9%	14.9%		
8	17.4%	15.5%	15.1%	15.6%	15.4%	14.8%	15.2%	15.2%	15.7%

VFA	9.5 mm			12.5 mm			19 mm		
	F	D	C	F	D	C	F	D	C
4	33.3%	36.3%	36.6%	41.3%	46.4%	42.2%	51.7%	48.0%	54.4%
4	32.0%	36.2%	34.9%	43.5%	47.2%	43.5%	52.9%	49.9%	46.7%
5	46.4%	52.2%	52.3%	61.6%	71.1%	63.9%	75.4%	68.5%	79.2%
5	44.1%	54.3%	53.1%	58.6%	71.9%	66.4%	80.9%	66.3%	72.3%
6	60.3%	74.4%	74.0%	75.3%	95.2%	88.9%	93.6%	87.3%	95.4%
6	61.4%	69.0%	73.2%	80.7%	98.7%	89.6%	96.1%	91.3%	92.0%
7	74.2%	89.5%	95.3%	95.4%	104.2%	98.1%	101.4%	96.1%	90.7%
7	75.8%	87.7%	95.3%	92.8%	102.4%	96.4%	100.6%	88.6%	97.0%
8	91.7%	95.2%	99.4%	101.0%	103.0%	98.7%	103.5%		
8	91.4%	97.1%	98.8%	101.7%	101.8%	99.3%	101.3%	96.6%	90.9%

Dust-P_{be} Ratio	9.5 mm			12.5 mm			19 mm		
	F	D	C	F	D	C	F	D	C
4	1.41	1.47	1.71	1.40	1.56	1.78	1.52	1.75	1.82
4	1.41	1.47	1.71	1.40	1.56	1.78	1.52	1.75	1.82
5	1.04	1.08	1.20	1.04	1.12	1.23	1.10	1.21	1.24
5	1.04	1.08	1.20	1.04	1.12	1.23	1.10	1.21	1.24
6	0.82	0.85	0.92	0.82	0.88	0.94	0.86	0.93	0.94
6	0.82	0.85	0.92	0.82	0.88	0.94	0.86	0.93	0.94
7	0.68	0.70	0.75	0.68	0.72	0.76	0.71	0.75	0.76
7	0.68	0.70	0.75	0.68	0.72	0.76	0.71	0.75	0.76
8	0.58	0.60	0.63	0.58	0.61	0.64	0.60	0.63	0.64
8	0.58	0.60	0.63	0.58	0.61	0.64	0.60	0.63	0.64

Table A.3 ***Continued***

K	9.5 mm			12.5 mm			19 mm		
	F	**D**	**C**	**F**	**D**	**C**	**F**	**D**	**C**
4	2.55	2.51	2.45	2.50	2.45	2.40	2.48	2.43	2.39
5	3.22	3.17	3.09	3.16	3.10	3.03	3.14	3.06	3.02
6	3.91	3.84	3.75	3.83	3.76	3.68	3.81	3.72	3.66
7	4.61	4.53	4.42	4.51	4.43	4.34	4.49	4.38	4.31
8	5.32	5.23	5.11	5.22	5.12	5.01	5.18	5.06	4.98

Film Thickness	9.5 mm			12.5 mm			19 mm		
	F	**D**	**C**	**F**	**D**	**C**	**F**	**D**	**C**
4	4.2	4.5	4.6	4.7	4.7	4.7	4.4	4.4	4.7
5	5.7	6.2	6.6	6.3	6.6	6.9	6.4	6.4	6.8
6	7.2	7.9	8.7	8.0	8.5	9.0	8.4	8.4	9.0
7	8.7	9.6	10.7	9.7	10.3	11.1	10.4	10.4	11.2
8	10.2	11.3	12.7	11.3	12.2	13.3	11.3	12.4	13.4

Table A.4 Volumetric Results for 100 Percent Natural Specimens

Property	9.5 mm			12.5 mm			19 mm		
	F	D	C	F	D	C	F	D	C
G_{sb}	2.580	2.559	2.542	2.565	2.554	2.544	2.560	2.549	2.543
SA	6.65	5.95	4.98	5.99	5.37	4.68	5.80	5.02	4.56
G_{se}	2.667	2.643	2.642	2.641	2.648	2.635	2.672	2.680	2.687
Abs (%)	1.29	1.27	1.52	1.15	1.42	1.39	1.67	1.95	2.16

G_{mb}	9.5 mm			12.5 mm			19 mm		
	F	D	C	F	D	C	F	D	C
4	2.286	2.312	2.329	2.328	2.360	2.315	2.365	2.403	2.404
4	2.274	2.310	2.328	2.316	2.349	2.307	2.360	2.406	2.410
5	2.307	2.352	2.378	2.355	2.414	2.333	2.394	2.436	2.442
5	2.336	2.363	2.372	2.348	2.391	2.349	2.407	2.439	2.442
6	2.319	2.368	2.385	2.385	2.405	2.384	2.408	2.440	2.437
6	2.350	2.376	2.395	2.377	2.401	2.379	2.414	2.438	2.437
7	2.331	2.373	2.371	2.376	2.389	2.364	2.404	2.398	2.406
7	2.336	2.369	2.384	2.368	2.377	2.380	2.396	2.394	2.399
8	2.336	2.343	2.343	2.347	2.345	2.336	2.364	2.365	
8	2.337	2.342	2.354	2.353	2.348	2.325	2.368	2.374	

G_{mm}	9.5 mm			12.5 mm			19 mm		
	F	D	C	F	D	C	F	D	C
4	2.503	2.486	2.486	2.484	2.491	2.480	2.511	2.519	2.524
4	2.503	2.486	2.486	2.484	2.491	2.480	2.511	2.519	2.524
5	2.465	2.449	2.449	2.448	2.454	2.444	2.474	2.482	2.487
5	2.465	2.449	2.449	2.448	2.454	2.444	2.474	2.482	2.487
6	2.428	2.414	2.414	2.413	2.419	2.409	2.437	2.446	2.450
6	2.428	2.414	2.414	2.413	2.419	2.409	2.437	2.446	2.450
7	2.392	2.380	2.380	2.378	2.384	2.375	2.402	2.411	2.414
7	2.392	2.380	2.380	2.378	2.384	2.375	2.402	2.411	2.414
8	2.358	2.346	2.347	2.345	2.351	2.342	2.368	2.376	
8	2.358	2.346	2.347	2.345	2.351	2.342	2.368	2.376	

Air Voids	9.5 mm			12.5 mm			19 mm		
	F	D	C	F	D	C	F	D	C
4	9.0%	7.0%	6.3%	6.3%	5.3%	6.7%	5.8%	4.6%	4.8%
4	9.5%	7.1%	6.3%	6.8%	5.7%	7.0%	6.0%	4.5%	4.5%
5	7.3%	4.0%	2.9%	3.8%	1.6%	4.5%	3.2%	1.9%	1.8%
5	6.7%	3.5%	3.2%	4.1%	2.6%	3.9%	2.7%	1.7%	1.8%
6	4.2%	1.9%	1.2%	1.2%	0.6%	1.0%	1.2%	0.2%	0.5%
6	4.9%	1.6%	0.8%	1.5%	0.7%	1.2%	0.9%	0.3%	0.5%
7	3.0%	0.3%	0.4%	0.1%	-0.2%	0.5%	-0.1%	0.5%	0.3%
7	2.8%	0.4%	-0.2%	0.4%	0.3%	-0.2%	0.3%	0.7%	0.7%
8	1.3%	0.1%	0.2%	-0.1%	0.2%	0.2%	0.2%	0.5%	
8	1.3%	0.2%	-0.3%	-0.3%	0.1%	0.7%	0.0%	0.1%	

Table A.4 *Continued*

VMA	9.5 mm			12.5 mm			19 mm		
	F	D	C	F	D	C	F	D	C
4	14.9%	13.3%	12.1%	12.9%	11.3%	12.6%	11.3%	9.5%	9.2%
4	15.4%	13.3%	12.1%	13.3%	11.7%	12.9%	11.5%	9.4%	9.0%
5	15.5%	12.7%	11.1%	12.8%	10.2%	12.9%	11.2%	9.2%	8.8%
5	15.0%	12.3%	11.4%	13.0%	11.1%	12.3%	10.7%	9.1%	8.8%
6	14.9%	13.0%	11.8%	12.6%	11.5%	11.9%	11.6%	10.0%	9.9%
6	15.5%	12.7%	11.4%	12.9%	11.6%	12.1%	11.4%	10.1%	9.9%
7	16.0%	13.8%	13.3%	13.9%	13.0%	13.6%	12.7%	12.5%	12.0%
7	15.8%	13.9%	12.8%	14.2%	13.4%	13.0%	13.0%	12.7%	12.3%
8	16.7%	15.8%	15.2%	15.8%	15.5%	15.5%	15.1%	14.6%	
8	16.7%	15.8%	14.8%	15.6%	15.4%	15.9%	14.9%	14.3%	

VFA	9.5 mm			12.5 mm			19 mm		
	F	D	C	F	D	C	F	D	C
4	39.9%	47.2%	47.7%	51.1%	53.5%	47.3%	48.7%	51.6%	48.5%
4	38.4%	46.9%	47.6%	49.1%	51.4%	46.0%	47.8%	52.2%	49.9%
5	53.1%	68.7%	73.8%	70.3%	84.1%	64.8%	71.1%	79.9%	79.5%
5	55.2%	71.3%	72.2%	68.8%	76.8%	68.4%	74.8%	81.3%	79.7%
6	72.0%	85.5%	89.7%	90.8%	94.9%	91.3%	89.6%	97.7%	94.6%
6	68.6%	87.7%	93.1%	88.5%	93.7%	89.8%	91.7%	96.9%	94.6%
7	81.5%	98.1%	97.1%	99.3%	101.4%	96.6%	100.6%	96.0%	97.1%
7	82.6%	97.0%	101.3%	96.9%	97.8%	101.7%	98.0%	94.5%	94.7%
8	92.0%	99.3%	98.9%	100.5%	98.4%	98.4%	98.8%	96.9%	
8	92.1%	98.9%	102.1%	102.2%	99.2%	95.4%	99.9%	99.4%	

Dust-P_{be} Ratio	9.5 mm			12.5 mm			19 mm		
	F	D	C	F	D	C	F	D	C
4	1.5	1.5	1.6	1.4	1.5	1.5	1.7	2.0	2.1
4	1.5	1.5	1.6	1.4	1.5	1.5	1.7	2.0	2.1
5	1.1	1.1	1.1	1.0	1.1	1.1	1.2	1.3	1.4
5	1.1	1.1	1.1	1.0	1.1	1.1	1.2	1.3	1.4
6	0.9	0.8	0.9	0.8	0.9	0.9	0.9	1.0	1.0
6	0.9	0.8	0.9	0.8	0.9	0.9	0.9	1.0	1.0
7	0.7	0.7	0.7	0.7	0.7	0.7	0.8	0.8	0.8
7	0.7	0.7	0.7	0.7	0.7	0.7	0.8	0.8	0.8
8	0.6	0.6	0.6	0.6	0.6	0.6	0.6	0.7	
8	0.6	0.6	0.6	0.6	0.6	0.6	0.6	0.7	

Table A.4 ***Continued***

K	9.5 mm			12.5 mm			19 mm		
	F	D	C	F	D	C	F	D	C
4	2.52	2.47	2.41	2.47	2.42	2.37	2.45	2.40	2.36
5	3.18	3.13	3.05	3.12	3.06	3.00	3.10	3.03	2.98
6	3.86	3.79	3.70	3.78	3.71	3.64	3.76	3.67	3.62
7	4.55	4.47	4.36	4.46	4.38	4.29	4.43	4.33	4.27
8	5.25	5.16	5.04	5.15	5.06	4.95	5.12	5.00	

Film Thickness	9.5 mm			12.5 mm			19 mm		
	F	D	C	F	D	C	F	D	C
4	4.1	4.6	5.0	4.8	4.8	5.6	4.0	4.1	4.0
5	5.6	6.3	7.0	6.4	6.7	7.7	5.7	6.1	6.2
6	7.1	7.9	9.0	8.1	8.5	9.9	7.5	8.1	8.4
7	8.6	9.6	11.0	9.8	10.4	12.0	9.2	10.1	10.6
8	10.1	11.3	13.0	11.4	12.3	14.1	10.9	12.0	

APPENDIX B

NOTTINGHAM ASPHALT TESTER RESULTS

Table B.1 Accumulated Axial Microstrain at 1800 Cycles for 100 Percent Crushed Specimens

Asphalt Content (%)	Specimen ID	9.5C	12.5C	19C	9.5D	12.5D	19D	9.5F	12.5F	19F
4	1	9462	9645	9645	7729	8545	8510	8101	8690	7331
4	2	9974	8617	9494	8234	8516	8267	9640	8152	8669
5	1	9822	9254	9758	8049	9508	10408	7606	8358	7475
5	2	9228	9468	9264	8207	8988	8069	9696	8057	8128
6	1	9130	9546	10347	8819	8352	9876	8257	8165	8158
6	2	8984	9532	11315	8748	8004	10696	8916	8719	8492
7	1	11920	16206	14205	9552	14669	17169	8193	8282	8974
7	2	9250	14845	12979	9392	12520	18699	9450	6296	9456
8	1	21091	27823	29365	18880	24661	30125	8692	21515	23615
8	2	28868	33258	27849	20559	21188	34319	8297	17029	20653

Table B.2 Stiffness (kPa) at 1800 Cycles for 100 Percent Crushed Specimens

Asphalt Content (%)	Specimen ID	9.5C	12.5C	19C	9.5D	12.5D	19D	9.5F	12.5F	19F
4	1	416474	451482	451340	406157	430875	462183	347307	407752	431472
4	2	422562	463575	451482	386504	415363	418794	334382	400414	411522
5	1	404934	460068	426515	400278	423545	437259	365337	399758	426515
5	2	403323	443302	442657	426515	416582	445032	340021	396157	414312
6	1	379882	436631	457146	402788	405394	447297	363169	401194	392394
6	2	385124	430219	432123	388881	404847	401892	360558	399088	395455
7	1	384993	379507	420343	371790	401472	396454	336041	380731	380858
7	2	391926	398611	391161	375554	389600	355558	336785	421432	388844
8	1	333357	332237	357025	340629	355315	354466	379657	331866	318677
8	2	307768	319363	339072	360934	352780	334485	342461	353621	329653

Table B.3 Accumulated Axial Microstrain at 1800 Cycles for 50 Percent Crushed/50% Natural Specimens

Asphalt Content (%)	Specimen ID	9.5C	12.5C	19C	9.5D	12.5D	19D	9.5F	12.5F	19F
4	1	10992	9597	11081	11380	8570	9404	12467	10343	9089
4	2		10865	10400	9168	8781	8938	11446	9819	8320
5	1	12523	12621	9524	10241	10419	10168	11080	10964	9755
5	2	11846	11765	10093	8991	9235	9570	10599	11581	9508
6	1	12059	10406	17080	10793	10347	15402	11925	11238	15625
6	2	12443	10016	17907	11021	13950	20918	11267	10799	15415
7	1	11088	22742	37021	10680	31132	48486	12176	12526	37860
7	2	11932	24185	42603	9876	26056	37240	10529	15897	39707
8	1	29083	49430		28016	61633		13934	34376	62955
8	2	29179	48128		26620	59107	70514	13940	36540	72204

Table B.4 Stiffness (kPa) at 1800 Cycles for 50 Percent Crushed/50 Percent Natural Specimens

Asphalt Content (%)	Specimen ID	9.5C	12.5C	19C	9.5D	12.5D	19D	9.5F	12.5F	19F
4	1	388216	354624	386411	349868	398876	418380	352809	374877	405941
4	2		374014	406015	354591	390869	404302	338577	362134	412633
5	1	363417	351161	381981	334944	365511	383481	323646	344790	377947
5	2	335309	350521	380988	355443	365072	368528	339323	335309	374175
6	1	313511	345495	369912	330469	358041	335971	345752	329021	342666
6	2	322145	348370	348939	315462	345681	318580	291142	333774	344277
7	1	316974	306785	293081	314955	288934	286673	288485	306496	279561
7	2	325316	294156	280845	312263	290079	268051	298558	276761	270776
8	1	261861	256207		260710	239651		291568	259570	254449
8	2	260710	258664		256429	249108	255986	285006	252931	255986

Table B.5 Accumulated Axial Microstrain at 1800 Cycles for Manufactured Fine-Natural Coarse Specimens

Asphalt Content (%)	Specimen ID	9.5C	12.5C	19C	9.5D	12.5D	19D	9.5F	12.5F	19F
4	1	10992	14524	14811	9743	11038	13235	7943	10736	10411
4	2		19698	12385	10127	11019	13669	8179	10473	10171
5	1	12523	12675	19088	11731	11149	11847	8603	12901	11241
5	2	11846	12025	14177	8906	11532		8338	12513	11036
6	1	12059	12032	15024	9098	10358	11988	8837	11367	11422
6	2	12443	11291	16792	9062	9719	12137	8707	11137	12515
7	1	11088	18469	33259	10618	24262	26485	9097	12201	17099
7	2	11932	20216	31923	10224	24004	28121	9818	11012	14190
8	1	29083	43253		22862	41988	52038	9527	21456	30816
8	2	29179	40346		24057	42817	48077	9987	15296	34341

Table B.6 Stiffness (kPa) at 1800 Cycles for Manufactured Fine-Natural Coarse Specimens

Asphalt Content (%)	Specimen ID	9.5C	12.5C	19C	9.5D	12.5D	19D	9.5F	12.5F	19F
4	1	388216	377686	412626	361395	382853	389379	358100	370424	414285
4	2		349788	413200	376690	398723	418878	375609	385368	397692
5	1	363417	362798	354466	371302	380731	375966	367831	371302	362894
5	2	335309	363676	372256	372634	381214		359033	374426	387395
6	1	313511	373192	383984	382522	388335	377065	360976	357093	402562
6	2	322145	368099	380493	368166	370860	341468	356841	351386	372256
7	1	316974	337145	317183	371790	326621	345088	352190	345681	356423
7	2	325316	323734	315711	349868	335601	341468	344311	350113	354697
8	1	261861	285827		301819	298179	294042	337145	311320	315626
8	2	260710	291156		308087	285003	297283	332355	323542	308408

Table B.7 Accumulated Axial Microstrain at 1800 Cycles for 100 Percent Natural Specimens

Asphalt Content (%)	Specimen ID	9.5C	12.5C	19C	9.5D	12.5D	19D	9.5F	12.5F	19F
4	1	10758	14403	15315	12607	9863	12183	14351	11907	14218
4	2	12396	12273	16147	12489	9582	9667	15480	15004	17262
5	1	10891	12107	11832	12321	12191	13035	14677	13738	13489
5	2	10101	12526	13073	13174	12194	11623	19689	11222	14664
6	1	18232	11021	21302	12779	17131	21202	19519	15176	18905
6	2	16199	11583	23555	14461	25865	26812	19767	14962	15966
7	1	27495	27204	44417	44819	55150	73594	21338	43538	57230
7	2	29094	25779	38136	36508	61953	79839	19788	57563	50712
8	1	56096	43946		90560			46030	76767	
8	2	65151	59604		88263			42281	84825	

Table B.8 Stiffness (kPa) at 1800 Cycles for 100 Percent Natural Specimens

Asphalt Content (%)	Specimen ID	9.5C	12.5C	19C	9.5D	12.5D	19D	9.5F	12.5F	19F
4	1	331339	350419	345967	313456	362894	339460	306315	353611	327944
4	2	345858	319707	361054	316031	344790	365891	291961	321981	310946
5	1	310327	297692	332931	324899	330388	323190	346907	320448	343787
5	2	323172	297405	337534	343703	325457	345088	256940	317769	327470
6	1	264900	309687	306756	331495	306813	300697	248452	291442	282291
6	2	273445	286912	275234	290519	280423	287546	262864	297717	295342
7	1	267019	248691	251269	238979	232146	209970	244851	237656	237350
7	2	239742	256969	274069	235790	225790	224594	261483	200532	245197
8	1	213258	223343		194690			209583	213108	
8	2	194316	210068		222625			224101	199070	